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Journal of the
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THE LEMOLO NO. 1 ROCKFILL DAM

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SYNOPSIS

This paper describes the Lemolo No. 1 Rockfill Dam of the California Oregon Power Company. The dam, which was built during the winter of 1953 - 1954 and the spring of 1954, is a deck type with a reinforced concrete deck. The reservoir was partially filled in 1954 and was filled in 1955. The dam is relatively small and is of interest primarily because the north abutment was founded on soil consisting of a mixture of old talus, ancient streambed material and clay. A deck type rockfill dam founded on soil is unusual, if not unique.

General Description of Development

The Lemolo No. 1 Dam is the storage and diversion dam for the Lemolo No. 1 hydro-electric development of the California Oregon Power Company. The dam is located on the North Umpqua River about 25 miles northwest of Crater Lake at an elevation of 4000 feet above sea level. Its purpose is to divert water into the opwn power conduit leading to the power plant and to provide 12,000 acre-ft of seasonal storage of the Lemolo No. 1 development and for four downstream developments. The water regulated by this dam falls through a net effective head of 2140 feet in the five plants served by it. The North Umpqua River has a drainage area of 175 miles and an average flow of 394 cfs at the damsite.

The dam is a deck type rockfill having a reinforced concrete deck. (See Fig. 1) It has a maximum height of 120 feet and a length of 775 feet along the crest. Because of the soil foundation on the north abutment, unusual measures were taken to protect the north abutment from erosion and extra

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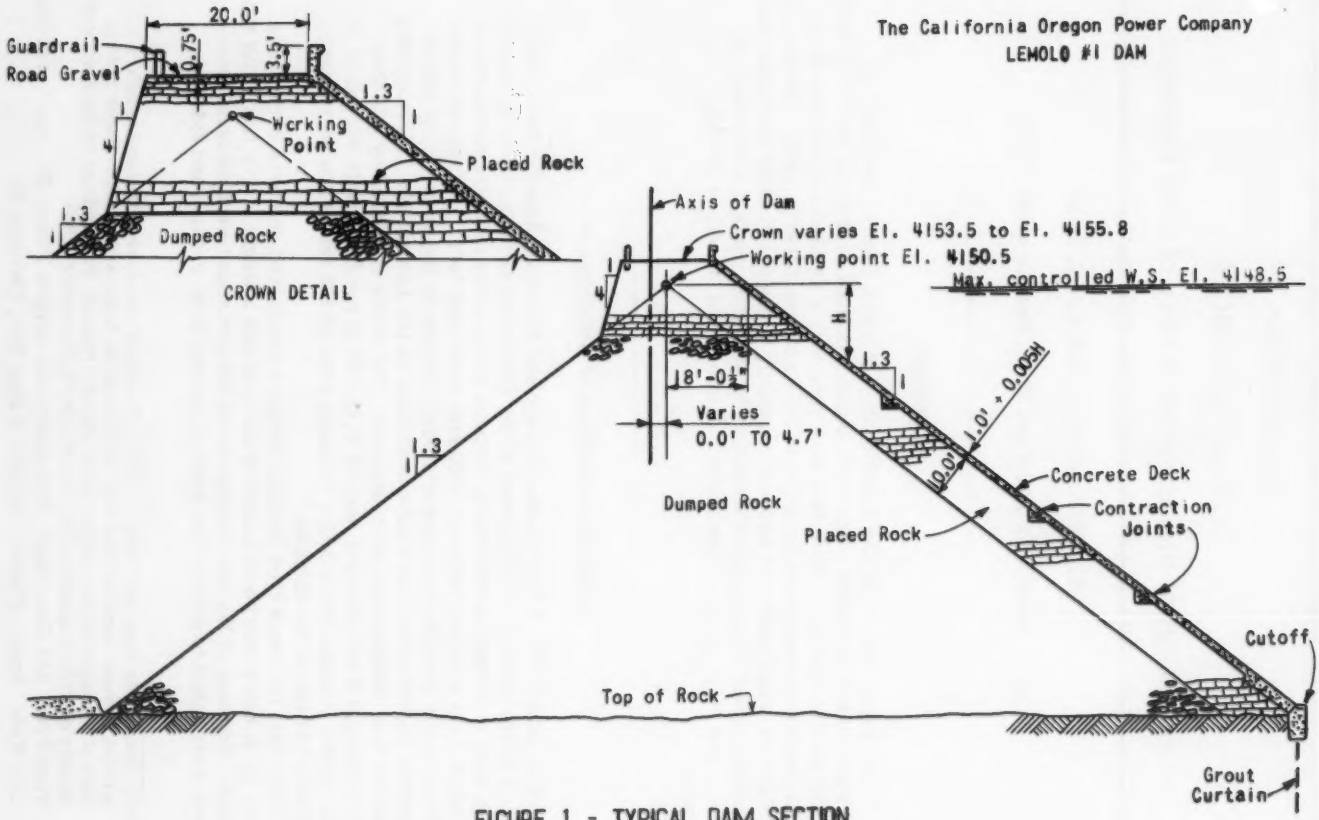

FIGURE 1 - TYPICAL DAM SECTION

precautions were taken in the construction of the concrete deck to prevent cracking and the accompanying leakage. Excessive leakage around the cutoff or through the deck, or heavy precipitation flowing down through the loose rockfill could become concentrated and erode the soil foundation of the north abutment and cause failure or severe damage, if not prevented or collected and safely carried out of the fill.

A rockfill was selected because the foundation was not considered suitable for a concrete dam and material for compacted fill could not be located within economical hauling distance. The deck type was selected because material for an impervious compacted membrane could not be located within economical hauling distance. As is usual, economic considerations played an important role in the selection of the type of dam. In this case, however, local conditions made it unnecessary to prepare comparative estimates to select the most economical type which could be depended upon to safely perform its function.

Outlets

Two outlets for the controlled release of water were provided. A low level sluice capable of releasing water for downriver developments when the power plant is shut down, was located at streambed level. The power outlet designed to control flow to the open power conduit was located on the north abutment at the highest level which would permit releases when the storage reservoir was drawn to its lowest operating level.

The low level sluice conduit is a 9 feet diameter cast in place reinforced concrete pipe through the fill which was used for stream diversion during construction. After diversion was completed, a 60 inch Howell-Bunger valve was installed at the lower end of the conduit. The Howell-Bunger valve was protected by a 66 inch butterfly valve. A 12 inch Howell-Bunger valve was installed to return water to the streambed for fish. The intake is equipped with trashracks and a slide type gate which can be used to unwater the conduit through the dam.

The power outlet conduit is a 7 feet 6 inches diameter cast in place reinforced concrete pipe through the fill. Flow is controlled at the downstream end by two 60 inch Howell-Bunger valves. The intake is equipped with trashracks and a wheeled gate capable of closing against full flow through the conduit.

Spillway

The spillway was excavated through the rock on the south abutment and consists of a channel about 800 feet long with a control section located on the axis of the dam. The control section was lined with concrete, but the remainder of the rock channel was not lined. The control section consists of three 33 feet 4 inch long bays separated by piers. The two outboard bays are equipped with 3 feet high flashboards and the center bay is equipped with a 10 feet high tainter gate. The spillway crests are located to provide a discharge capacity of 16000 cfs with 3 feet of freeboard on the dam. Opening the low level sluice and the power outlet will increase the discharge capacity to 18000 cfs. Rock from the spillway excavation meeting the requirements for fill material was used in the fill.

FIGURE 1 - TYPICAL DAM SECTION

Foundation Explorations

The foundation was explored by making a few core drillings at strategic points and digging a number of surface exploration trenches. Except for the upper part of the north abutment, the rock was exposed or covered with only a few feet of overburden. Anyone familiar with the extremely irregular formations in this volcanic area will appreciate that a great many core borings are unwarranted as the information produced soon reaches the point of diminishing returns.

Geology

All the rocks in the region have been formed directly or indirectly by volcanic action. The formations are characterized by their extreme irregularity produced by the effects of intermittent lava flows, stream erosion and deposition, and ash deposits.

As exposed by the exploration program, the foundation stripping and the cutoff excavation, the foundation at the streambed and part way up the abutments consists of alternate layers of dense hard basalt and basaltic agglomerate. (See Fig. 2). On the south abutment beginning at about 60 feet above streambed there is an 18 feet layer of well cemented laminated impervious tuff. Above the tuff the alternate layers of agglomerate and basalt continue well above the top of the dam.

On the north abutment beginning about 40 feet above streambed there is a 16 feet layer laminated tuff of low permeability but which is not as well cemented as that on the south side.

Above the tuff there is only soil consisting of a mixture of old talus, ancient streambed material, and basalt boulders mixed with clay and having a low permeability. The photograph Fig. 3 shows the north abutment after the stripping was completed and excavation of the cutoff started. The dark nearly horizontal strata in the center of the picture is the tuff. The lighter material above the tuff is soil. A few boulders embedded in the soil are visible.

Rock Fill

The specifications for the rock to be used in the dam required that it should be hard and durable and that it should be free from cracks or incipient jointing which would cause it to shatter when dumped in the fill. They also required that it should resist crushing under load and be of such a mineral composition as to resist disintegration under the action of the elements. The specifications further required that the grading be such as to produce a dense fill with rock sizes ranging from 1 to 10 tons predominating in the fill. It was further required that not more than five percent of the fill should consist of fines less than 4 inches in size.

The rock used in the fill is all basalt and was obtained from a quarry on the south abutment about 1/2 mile downstream from the dam, and from the spillway excavation. The fill is of conventional design and was built in two zones consisting of a dumped fill making up the main body of the dam, and a more dense placed fill upon which the concrete deck rests.

FIGURE 2 - GEOLOGIC SECTION ALONG CUTOFF

The California Oregon Power Co.
LEMOLO #1 DAM

Fig. 4 is a photograph of the upstream face of the dam during construction of the placed rock fill and drilling for the grout curtain. Construction of the power outlet control tower is also in progress. Fig. 5 is a photograph of the upstream face of the rock fill in a more advanced stage. In the left foreground can be seen the sluice outlet tower and the timber crib division structure.

Figure 3 - North Abutment

Figure 4 - Placing Rock Fill

The dumped fill was placed in three lifts. The conduits through the fill were protected by covering them with a minimum of 30 inches of quarry fines covered with 4 feet of placed rock before starting dumping operations. All dumped fill was sluiced as it was dumped, using three volumes of water to each volume of rock.

The 10 feet thick placed rock fill was constructed of selected large stones carefully placed and chinked to obtain the maximum contact between placed stones and between the placed fill and the dumped fill. After the placed fill was completed to grade the surface voids were filled with gravel to the maximum extent possible in order to minimize loss of cement when placing the deck.

In the area where the fill was placed in rock foundation, most of the loose overburden material and all the organic matter was removed. On the north abutment where the fill was placed on soil, the surface material was stripped to an average depth of ten feet to remove all stumps, roots and loosely compacted surface materials.

The dam was built with a slight camber upstream and vertically. The canyon was nearly "V" shaped and both the horizontal and the vertical cambers were laid out with straight lines intersecting at a point at the center of the dam. The horizontal camber was 4.7 feet and the vertical camber was 2.3 feet.

Cut-Off Wall

When in contact with rock, the cut-off wall has a minimum thickness of 3 feet and was keyed into the rock a minimum of 5 feet. The wall was permitted

Figure 5 - Rock Fill

to project 3 feet above the rock surface without special treatment. For projections exceeding 3 feet, the voids in the placed rock were filled with sand cement grout to form a lean rubble mass in contact with the cut-off wall.

On the north abutment where the wall did not extent to rock, its thickness was increased to a minimum of 4 feet. Three criteria were established to determine the bottom of cut-off.

1. When terminated in the tuff, the minimum depth of cut-off below the stripped foundation was 0.375 times the head on the foundation surface.
2. When terminated above the tuff the minimum depth of cut-off was equal to the head on the stripped foundation surface.
3. When the cut-off was extended into the foundation a distance less than the head on the stripped foundation, it was required to penetrate a minimum of 5 feet into the tuff.

These requirements established a definite minimum line for the bottom of the cut-off and provided a vertical creep distance of 0.75 times the head or a weighted creep ratio of 2.25 in the tuff. In the soil the vertical creep distance was twice the head, giving a weighted creep ratio of 6. As a further precaution against piping, the soil foundation was covered with an inverted filter blanket.

Grout holes were generally drilled and grouted from the bottom of the cut-off trench. The group curtain was extended down a distance equal to the head on the top of the hole. Spacing of holes was determined by field conditions and the grout take of adjacent holes. A total of 78 holes were grouted giving an average hole spacing of 11 feet. The average hole took 34 sacks of cement and individual holes varied from 6 to 302 sacks of cement. A number of holes were stage grouted to minimize surface leakage.

Concrete Deck

The concrete deck, details of which are shown on Figs. 6 and 7 was 12 inches thick at the top and increased in thickness 0.005 feet for each foot of

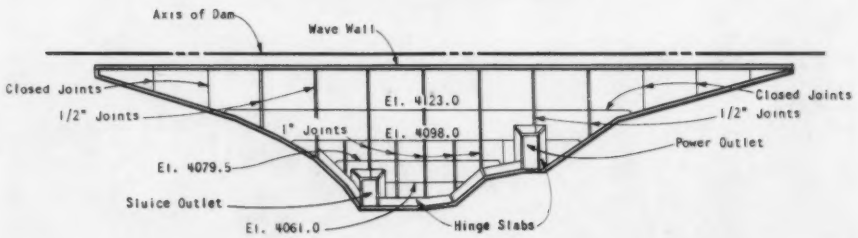

FIGURE 6 - PLAN OF CONCRETE DECK

The California Oregon Power Co.
LEMOLO #1 DAM

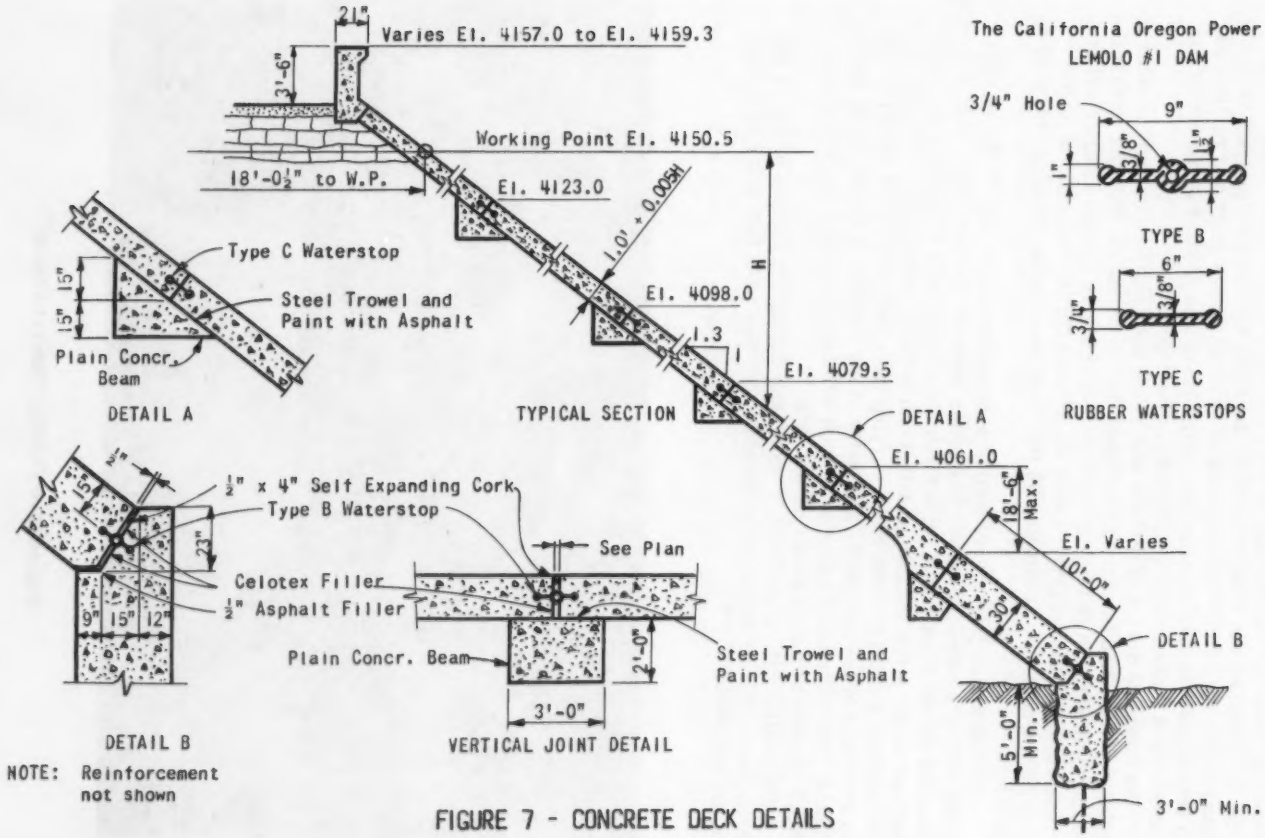

FIGURE 7 - CONCRETE DECK DETAILS

height. This resulted in a deck thickness of about 18 inches at the bottom.

The deck was reinforced with 0.50 percent reinforcement in both directions in the center of the slab. In the lower portions of the dam a 10 feet hinge slab spanning from the cut-off wall to the rock fill was provided. Hinge slabs were also used around the conduit intakes, the slabs spanning from seats on the intakes to the rock fill. The hinge slabs were 30 inches thick and reinforced in both faces. Designed structural reinforcing was located in the bottom of the slab and temperature reinforcing in the top.

The concrete deck was divided into small sections by contraction joints for ease of placing and to control cracking. Below elevation 4098 the contraction joints formed the deck into squares 30 feet in a side or into smaller areas as required by the topography. Above elevation 4098 the contraction joints divided the deck into rectangles 60 feet long and varying in height from 40 to 50 feet.

All contraction joints were sealed with rubber water stops. Three basic types of joints were provided. The joints between the deck and cutoff wall and between the deck and outlet structures were filled with 1/2 inch joint filler and had water stops capable of withstanding large movements. The vertical contraction joints had joint filler varying from one inch at the center of the dam to zero at the abutments and water stops capable of taking large movements. The horizontal joints were cast without joint filler and had water stops capable of taking small movements.

The photograph labeled Fig. 8 was taken during the placing of one of the deck slabs. Fig. 9 shows the completed concrete deck just prior to closure of the diversion.

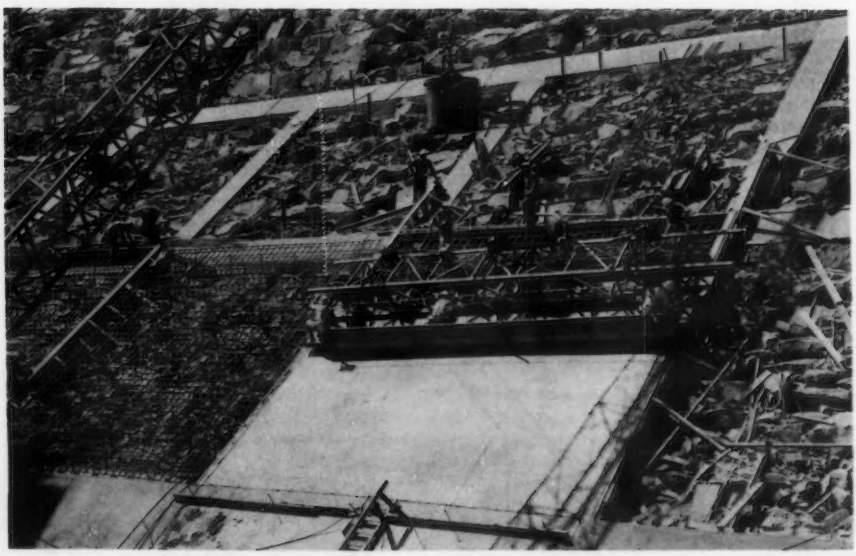

Figure 8 - Placing Concrete Deck

Drainage Provisions

A drainage system was designed to protect the soil on the north abutment from erosion which could be caused by excessive seepage around the cut-off, excessive leakage through the deck or from heavy precipitation running down through the fill. Water from any of these sources, if concentrated into rivulets running down the slope to the riverbed could erode the foundation and cause serious damage to the dam. The system was also useful in collecting sluicing water and transporting it out of the fill during rock placing operations.

The drainage system was divided into two parts. An inverted filter blanket was located under the placed rockfill and extended from the power outlet structure to the top of the dam. It was intended to intercept seepage and prevent piping around the cut-off. In addition, a series of french drains were constructed normal to the axis of the dam. They were designed to intercept and collect any water flowing on the surface of the foundation. Perforated pipe from the french drains was extended into the coarse gravel of the filter blanket and a collector drain at the toe of the rock fill carried all the water to a point where flow from the drainage system could be observed.

The inverted filter consisted of three 8 inch layers of graded material. The coarse sand placed in contact with the undisturbed foundation had a critical size of 1.0 mm. The fine gravel which followed had a critical size of 1/4 inch. The top layer consisted of coarse gravel having a critical size of 1-1/2 inches. During placing of the dumped rock fill the filter blanket was protected by a layer of placed rock having the voids filled with sand cement grout.

The french drains consist of perforated tile laid in trenches spaced at about 20 feet centers. The trenches were backfilled with coarse gravel which

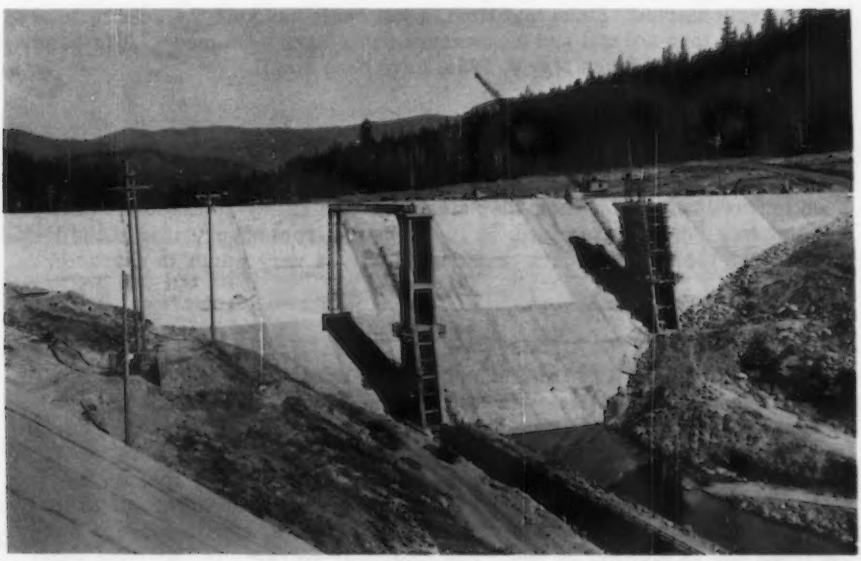

Figure 9 - Completed Concrete Deck

completely surrounds the perforated pipe. The trenches were filled with gravel to the ground surface and the rockfill placed directly on the gravel, thus insuring that any surface water can readily find its way to the drain tile.

The power conduit was located at the top of the hard rock and formed a barrier to the flow of water down the slope to the riverbed. To drain this pocket, the uphill slope of the trench excavated for the conduit was blanketed with gravel and a perforated tile drain was located at the bottom of the trench on the uphill side of the conduit.

Settlement

To provide permanent points for the measurement of displacements of the concrete deck, brass plugs were located at the four corners of each slab. Periodic measurements of the movements of these plugs were made between July 1, 1954, and May 6, 1955, covering the first cycle of reservoir filling and emptying. The maximum horizontal displacement of 0.245 feet was measured at a point about 35 feet below the crest of the dam. Displacements along a line 60 feet below the crest were of almost the same magnitude. The maximum settlement of 0.359 feet was measured at a point 60 feet below the crest. Settlements along a line 35 feet below the crest were of slightly smaller magnitude. Displacements above and below the two lines mentioned were smaller.

On Fig. 10 is plotted some of the results of the measurements which have been as consistent as could be expected. A typical settlement curve of a point on the high part of the fill is plotted against time for comparison with the reservoir level. Also included are the horizontal and vertical displacements along horizontal lines across the deepest part of the valley. These measurements were made on May 6, 1955, just before the second reservoir filling cycle started. Since that time, a wet cycle has kept the reservoir at a higher level than normal and no measurements have been made. It is believed that displacements since May 6, 1955, have been small.

Cracking of Concrete Deck

No cracks have been found in the concrete deck. The reservoir has not been lowered below elevation 4080 since the first filling so the lower part of the slab has not been examined. It is estimated from observation of the leakage through the dam that deck cracking, if any, is very minor in character.

Leakage

There has been a very limited amount of leakage through the north abutment which has been collected in the drainage system. There has been no leakage through the main dam structure. Such leakage between the two outlet conduits would show up in the stream bed at the toe of the dam.

There is a leak of about 1 cfs through an open seam in the rock on the south abutment. This leak appears in the surface near the lower end of the spillway channel excavation when the reservoir is full, and disappears when the reservoir is lowered 14 feet. The reservoir entrance to this open seam has not yet been located and it will be closed when uncovered.

FIGURE 10 - MOVEMENT OF FILL

There have been no unexplained increases or decreases in the leakage through the dam and it is believed that the cutoff and drainage provisions combined with the crack prevention program for the deck has produced satisfactory results.

ACKNOWLEDGMENTS

The Lemolo No. 1 Dam was designed by Pioneer Service and Engineering Co., under the direction of Mr. Stephen Wehner, who was then Chief Hydraulic Engineer. Construction was by the Morrison-Knudsen Construction Company.

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QUICK DESIGN OF AIR VENTS FOR POWER INTAKES

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ABSTRACT

For closed conduit systems without surge tanks, a common parameter is derived and correlated with vent diameter used in 35 hydroelectric installations. The resulting imperical equation for solution of vent diameter size requires knowledge of only rated head, output of turbine and length of vent. A graphic solution is also given.

In closed power conduits where a shut-off valve or gate is located at the head of the conduit, a vent, standpipe or air valve is always provided immediately downstream of the head gate. It fulfills three functions: (a) to reduce or eliminate subatmospheric pressure in the conduit during emergency closure of the head gate, (b) to facilitate drainage of the conduit; and (c) to allow air to escape when the conduit is filled. Sudden closure of the head gate in an emergency, such as occurrence of leaks in the penstock, and the consequent evacuation of water from the conduit, tends to create vacuum behind the gate and in the conduit, unless inflow of air is induced at a sufficient rate to balance the subatmospheric pressure. Excessive subatmospheric pressure can cause the collapse of a plate-steel penstock or the steel liner of a pressure tunnel.

If the intake is a massive concrete structure, it is most convenient to form the vent in the concrete. In some cases it may be desirable to embed a metal pipe in the concrete, thereby eliminating formwork. In other installations an unembedded metal standpipe may be used, but it would have to be anchored securely to the intake structure.

Most vents are circular in shape, but square and rectangular ones are often adopted where the vent also serves as an access shaft to the intake and

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conduit. In such cases, the size of the vent may be dictated by requirements other than that of air demand at the time of emergency closure. The vent inlet should be located sufficiently above the maximum reservoir surface, preferably on the downstream face of the intake, so that there is no interference with air flow. If it is also used as an access shaft, a bolted cover would be required at the top. An alignment with the minimum number of bends and sharp corners, and without abrupt changes in section is desirable for a vent. However, space limitations and other requirements may justify a vent that is not the most suitable from a theoretical standpoint.

Size of the air vent is normally based on the conservative assumption that air requirement with the head gate closed would be equal to the discharge of the turbine at full gate operation at normal head. Theoretically, the outflow diminishes as the head in the conduit reduces after closure, but due to the rapidity of sudden closure and because of the pumping action of the turbine, it is more correct to assume that there is no reduction in outflow. The U. S. Bureau of Reclamation, for example, recommends the design criterion that the rate of air inflow should be equal to the turbine discharge under full gate condition with head water on the conduit centerline at the intake and with the pressure behind the gate equal to half an atmosphere.

Where a surge tank is connected to the conduit, there is no appreciable flow through the air vent behind the head gate upon sudden closure at the intake, until the surge tank is drained. Furthermore, most of the air would be drawn into the surge tank if it is accessible to the atmosphere. The method of sizing air vents discussed herein is, therefore, not correctly applicable to conduit systems with surge tanks.

Under emergency closure condition, the extent of subatmospheric pressure allowed in penstocks is dependent upon the buckling strength of the conduits. If the penstock is encased or embedded in concrete, or provided with stiffener rings, its collapsing strength is much higher than that of an open unstiffened metal pipe. In any case, a certain amount of subatmospheric pressure is required for a differential head within the vent in order to induce sufficient flow of air into the conduit.

Cross-section of a typical power intake for a medium or high head installation is shown in Fig. 1. The power plant is not shown, but it is assumed that the penstock supplies a Francis or impulse type turbine, that a surge tank is not required and that the head gate is used for emergency closure. The following nomenclature is used:

d = diameter of the air vent, feet.

D = diameter of the penstock, feet.

f = Co-efficient of friction for air flow through the vent.

H = rated head of the turbine, feet.

L = Length of the air vent, feet.

P = rated output of the turbine, horsepower.

p_1 = pressure at air vent inlet (atmospheric), pounds per square foot.

p_2 = pressure in the penstock after emergency closure, pounds per square foot.

Q = Turbine discharge at rated output and head (full gate), cubic feet per second.

V = Velocity of air at vent outlet, feet per second.

w_1 = Unit weight of air at vent inlet, pounds per cubic foot.

w_2 = Unit weight of air at vent outlet, pounds per cubic foot.

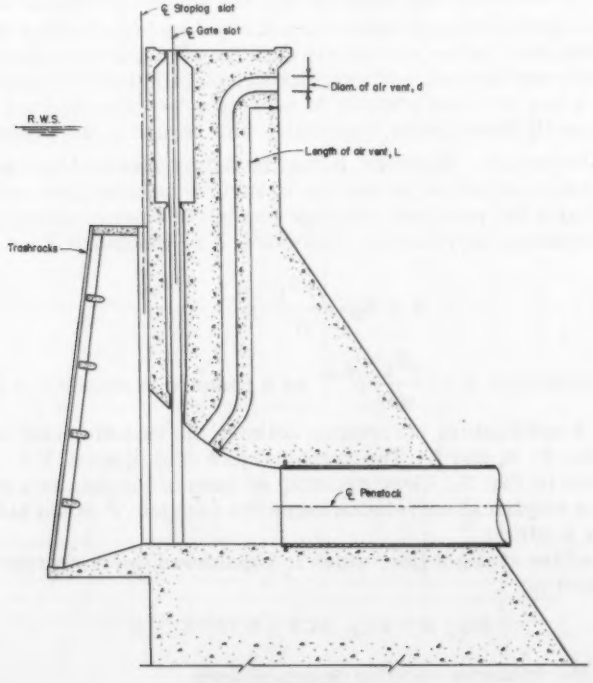
F, F_1, F_2 = Functions.

K_1, K_2, K_3 = Constants.

Assuming flow of air through the vent under either isothermal or adiabatic conditions, the diameter of the vent can be expressed as:

$$d = F_1 \left[V^2, L, f, \frac{w_1 w_2}{p_2 w_1 - p_1 w_2} \right]$$

Generally, air flow through vents is turbulent at maximum discharge, and Reynold's number exceeds 10^6. Therefore, the coefficient of friction, f, can be assumed constant for all air vents made of or formed in materials conventionally used for this purpose. Within the range of practical design, atmospheric pressure and density of air, p_1, w_1, w_2 are other factors that can be taken as constants. Hence,



TYPICAL POWER INTAKE

Fig. 1

$$d = F_1 (V^2, L, \frac{1}{p_2})$$

Flow of air through the vent at closure of the conduit is equal to or directly proportional to the turbine discharge. Therefore,

$$d = F_1 [\frac{Q^2}{d^4} , L, \frac{1}{p_2}]$$

This is further modified to:

$$d = F_2 [Q^{0.4}, L^{0.2}, p_2^{-0.2}]$$

or

$$d = K_1 [\frac{Q^2 L}{p_2}]^{0.2}$$

Discharge through the turbine is proportional to its output in horsepower, and inversely proportional to the corresponding head. Thus, the equation for diameter of the air vent becomes:

$$d = K_2 [\frac{P^2 \ L}{H^2 \ p_2}]^{0.2}$$

In order to determine the constant K_2, the above theoretical derivation was checked against data obtained from thirty-five installations listed in Table I. This information represents considerably different design practices in this country and abroad, and hydro-power installations of various sizes and types. It has not been possible to ascertain the criteria used for air vent design at all these plants, especially with regard to the pressure p_2 allowed in the conduit. However, it may be safely assumed that any differences between allowable pressures in various installations are small and their effect upon the resultant vent size negligible when considered in the above mathematical expression. Therefore, it is concluded that:

$$d = K_3 [\frac{P^2 L}{H^2}]^{0.2}$$

Using the function $F = [\frac{P^2 L}{H^2}]^{0.2}$ as a common parameter, it is possible to arrive at a satisfactory correlation between the vent diameter and other known factors, P, H, and L. The graphical plot of d against F to log-log scale is shown in Fig. 2. Close grouping of most of the data is a good indication that an empirical correlation using the function F would also be theoretically justified.

Equation of the straight line, which is considered fairly representative of the data plotted is:

$$\log_e F = \log_e 14.5 + 0.733 \log_e d.$$

From which the following equation is established:

$$d = 0.026\ F^{1.365} = 0.026\ \left[\frac{P^2\ L}{H^2}\right]^{0.273}$$

This equation is easy to use since it utilizes only those factors about which information is available at the design stage. It is believed that the results quickly obtained from this formula will enable the designer to avoid tedious calculations. As a precaution, however, the size of air vent obtained from this formula should be considered as the minimum recommended. If a vent section other than circular is desired, the equivalent area should be provided. In colder climates, where there is a possibility of icing in the vent, a larger size should be provided to compensate for the reduction in effective area.

To facilitate solution of the exponential terms, curves shown in Fig. 3 were prepared. With P, H, and L known, the minimum diameter of the air vent can be determined in a few minutes, and within a practical range of accuracy.

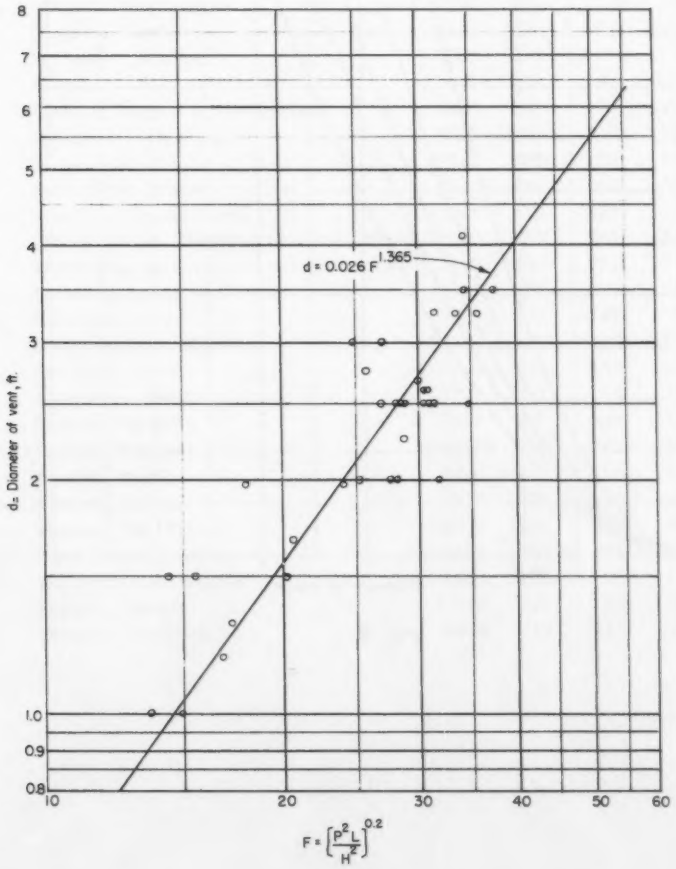


Fig. 2

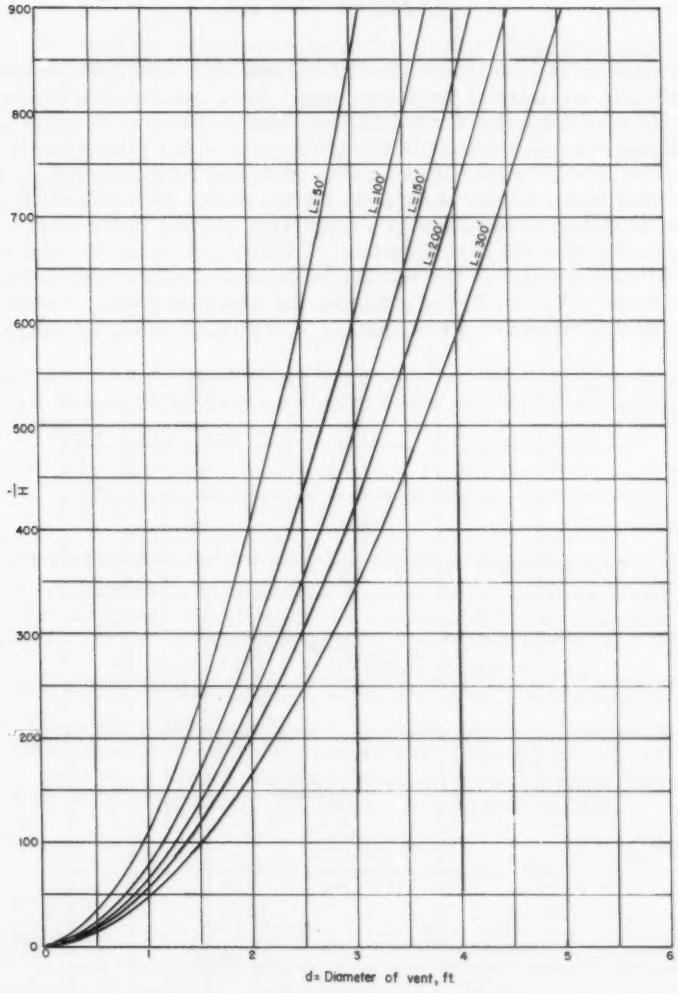

900
800
700
600
500
-|x
400
300
200
100
0
0
1
2
3
4
5
6
L= 50'
L= 100'
L= 150'
L= 200'
L= 300'
d= Diameter of vent, ft

Fig. 3

TABLE I

POWER INTAKE AIR VENT DATA

No.	Project	P 1000 hp	H ft	L ft	$\left[\frac{P^2 L}{H^2}\right]^{0.2}$	d ft
1	Beardsley, Calif.	14.0	258	250	14.9	1.0
2	Bhakra, India	125.0	370	275	31.5	2.5
3.	Bort, France	125.0	315	272	33.5	3.3
4.	Brownlee, Idaho	142.0	235	136	34.4	3.5
5.	Canyon Ferry, Mont.	23.5	125	97	20.2	1.5
6.	Chastang, France	134.0	233	198	35.8	3.3
7.	Cherokee, Tenn.	41.5	100	115	28.7	2.5
8.	Chief Joseph, Washington	100.0	165	54	28.7	2.3
9.	Clark Hill	55.0	136	90	27.0	3.0
10.	Davis, Arizona-Nevada	62.2	120	98	30.4	2.6
11.	Detroit, Washington	70.0	285	172	25.2	2.0
12.	Douglas, Tenn.	41.5	100	103	28.1	2.5
13.	Enchanet, France	39.3	178	167	24.0	2.0
14.	Falcon, U.S.A. - Mexico	14.75	110	80	16.8	1.2
15.	Flaming Gorge, Wyoming-Utah	50.0	365	185	20.6	1.7
16.	Fontana, Tenn.	91.5	330	246	28.4	2.5
17.	Furnas, Brazil	190.0	289	109	34.2	4.1
18.	Genissiat, France	90.0	210	164	31.3	3.3
19.	Glen Canyon, Arizona	155.5	450	225	30.5	2.6
20.	Grand Coulee, Washington(main units)	150.0	330	248	34.7	2.5
21.	Grand Coulee;(Station service unit)	14.0	330	257	13.6	1.0
22.	Hirakud, India	52.0	87	93	31.8	2.0
23.	Hiwassee, N.C.	80.0	190	147	30.3	2.5
24.	Hungry Horse, Montana	105.0	400	237	27.6	2.0
25.	Hwachon, Korea	2-37.0	250	132	25.8	2.8
26.	Itutinga, Brazil	17.0	97	52	17.3	1.3
27.	Kortes, Wyoming	18.5	200	125	16.0	1.5
28.	Nebeur, Tunisia	2--9.25	150	125	18.0	2.0
29.	Norris, Tenn.	66.0	165	180	30.9	2.5
30.	Rihand, India	53.0	225	140	23.8	3.0
31.	Shasta, California	103.0	330	255	30.0	2.7
32.	Tres Marias, Brazil	91.0	164	243	37.5	3.5
33.	Tulloch, California	12.0	153	98	14.3	1.5
34.	Waneta, Canada	120.0	210	55	28.1	2.0
35.	Yanhee, Thailand	96.8	328	175	26.8	2.5

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POWER FROM GEOTHERMAL STEAM AT THE GEYSERS POWER PLANT[a]

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SYNOPSIS

The generation of power from geothermal steam at the Geysers Power Plant is distinguished by the following features.

1. It will be the first commercial plant (12,500 KW) in the United States to generate power from geothermal steam and also will be the world's first privately-financed geothermal plant.
2. The condensed geothermal steam will provide the make-up water required for the evaporative cooling of the circulating water.
3. The turbine-generator will be installed at ground level.
4. The steam will be condensed in a barometric condenser and the non-condensable gases will be removed by large size steam jet gas ejectors.
5. The corrosive qualities of the circulating water will require the use of corrosion resistant alloy steels.
6. The plant will be designed to operate automatically unattended and with remote alarm to the manned Fulton Substation.

INTRODUCTION

In April 1847, William Bell Elliott, hunting grizzlies in the mountains between Cloverdale and Calistoga in California, came upon a narrow canyon

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a. Presented at the February 1959 ASCE Convention in Los Angeles, Calif.

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2. Whitman, Hoffman and Albritton, San Raphael, Calif.

which he thought surely must be the gate to Hades. Crowded between the barren, many hued banks of the defile for a distance of a quarter mile were a number of hot springs, fumaroles, and steam vents. "The Geysers," which were not geysers at all, had been discovered.

Although a few sightseers found their way to The Geysers in the following years, it was not until 1863 when Clark Foss opened a 26-mile stage line from Healdsburg to The Geysers, that they received many visitors. From the later 1860's to the early 1880's, a visit to The Geysers was a "must" for tourists to California, among whom were U. S. Grant, Theodore Roosevelt, Horace Greeley, Garibaldi, and William Jennings Bryan.

It was not until 1921 that an attempt was made to develop the area for the generation of power. Eight steam wells were drilled in the period from 1922-25. These are described in detail by E. T. Allen and Arthur L. Day in the Carnegie Institute of Washington's publication No. 378 of 1927.

Geothermal steam has been used in many places for heating. However, only in this century has it been used for generating electric power, first at Larderello in Tuscany, Italy, and in New Zealand in 1958. Now in Northern California, about 95 miles north of San Francisco, a 12,500 KW unit will be constructed to use the geothermal steam found at The Geysers in Big Sulphur Creek canyon in Sonoma County, whose location is shown in Fig. 1. However, mention should be made that for some time the Geysers Resort has operated a reciprocating steam engine driving a 15 KVA, 110 volt DC generator that supplies electricity to the resort.

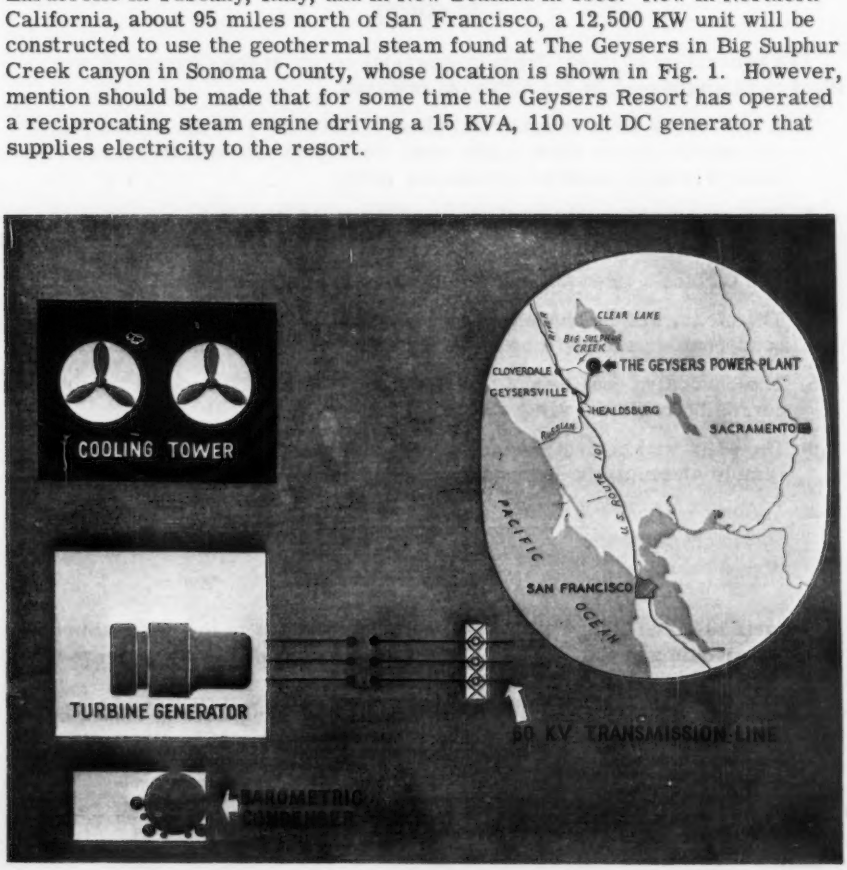


FIGURE 1 Location of Geyser Power Plant

Recent Development of the Geysers Area

Although steam wells were drilled in the early 1920's, further development was restrained by the abundant low cost hydro-electric resources and the relatively cheap fossil fuel available for steam-electric power generation. During the middle 1920's, The Geyser Development Company acquired the lands in Sonoma and Lake Counties on which the steam fumaroles are located. In 1955, the Magma Power Company obtained an exclusive lease of approximately 3200 acres for 99 years, renewable nine times, for the development and use of the steam on the property for the purpose of generating electric power. In October 1955, they drilled one well. In 1956, Magma Power Company made an agreement with the newly formed Thermal Power Company to share equally the benefits and expenses provided Thermal Power Company would drill additional wells and aid in marketing the steam.

From July to September of 1957, Thermal Power Company drilled five wells and redrilled Magma No. 1. In drilling Thermal No. 4, the steam blew out the nose of the hill. Thermal No. 3 did not develop enough steam so it was abandoned. The depth of the later steam wells is 523-1404 feet with the steam production at 500-700 feet. A general view of the wells is shown in Fig. 2, with all the wells blowing steam.

In December 1957, flow tests were made by Thermal Power Company on Magma No. 1, Thermal Nos. 1, 2 and 5. On the basis of these tests, Pacific Gas and Electric Company was approached with the proposal that it construct a steam-electric power plant with a transmission line which would connect the plant to its system. Feasibility and economic studies were made by P. G. and E. and the amount it could pay for the steam was determined. An

FIG. 2. THE GEYSERS

agreement was negotiated by the P. G. and E. with Magma Power Company and Thermal Power Company and was signed on July 16, 1958. On the basis of this agreement, a contract was prepared and signed on October 30, 1958, subject to California Public Utility Commission approval. This contract provides for installation by P. G. and E. of additional generating facilities when Magma and Thermal Companies have developed the necessary steam supply.

This plant will be the first commercial geothermal plant in the United States and also will be the world's first privately-financed geothermal plant.

Initial Studies of the Geothermal Steam

To determine what the special problems in the use of geothermal steam to generate power are, the experiences of the Italians and the reports from New Zealand were studied. It was soon realized that there are important differences in the steam at The Geysers.

First, the chemical contaminants in the steam in Italy are different from that reported by Allen and Day for The Geysers. In this respect The Geysers' steam wells are more favorable. In Italy, the steam contains boron which, although valuable as a chemical, adds complications from the corrosion viewpoint.

The important difference between the steam in New Zealand and that at The Geysers is that the latter is dry and slightly superheated. In the New Zealand fields, the steam originates from water under high pressure and temperature which partly flashes into steam when the pressure is reduced.

The information obtained from Allen and Day's report and a 1956 test of the gases in the steam were useful in the feasibility study. However, it was realized that in order to specify the correctly sized gas removing equipment and also in order to specify the least expensive materials necessary for this equipment, it would be necessary to make additional tests. These tests and the other tests listed below are being accomplished under the direction of the P. G. and E.'s Bureau of Tests and Inspections. The tests are listed in the chronological order in which they were started.

1. Tests of steam flow from Magma No. 1, Thermal No. 2 and Thermal No. 5 at varying pressures to determine the quantity and enthalpy.

2. Tests for the percentage of non-condensable gases in the steam. This is necessary for sizing the gas ejectors of the barometric condensers.

3. Tests for the wet-bulb temperatures of the air. These are required for the cooling tower specifications.

4. Corrosion tests for determining which materials should be used in various items of equipment.

5. Tests for the chemistry of the water in the cooling water system.

6. Test for the radioactivity of the steam condensate.

Test for Steam Flow

The first tests were made to determine the quantities of steam from the three wells which are planned to be used for supplying the turbine. The steam quantities increase with lower pressure at the wellhead as shown in Fig. 3. It is calculated that the wellhead pressures will be about 115 lbs. per

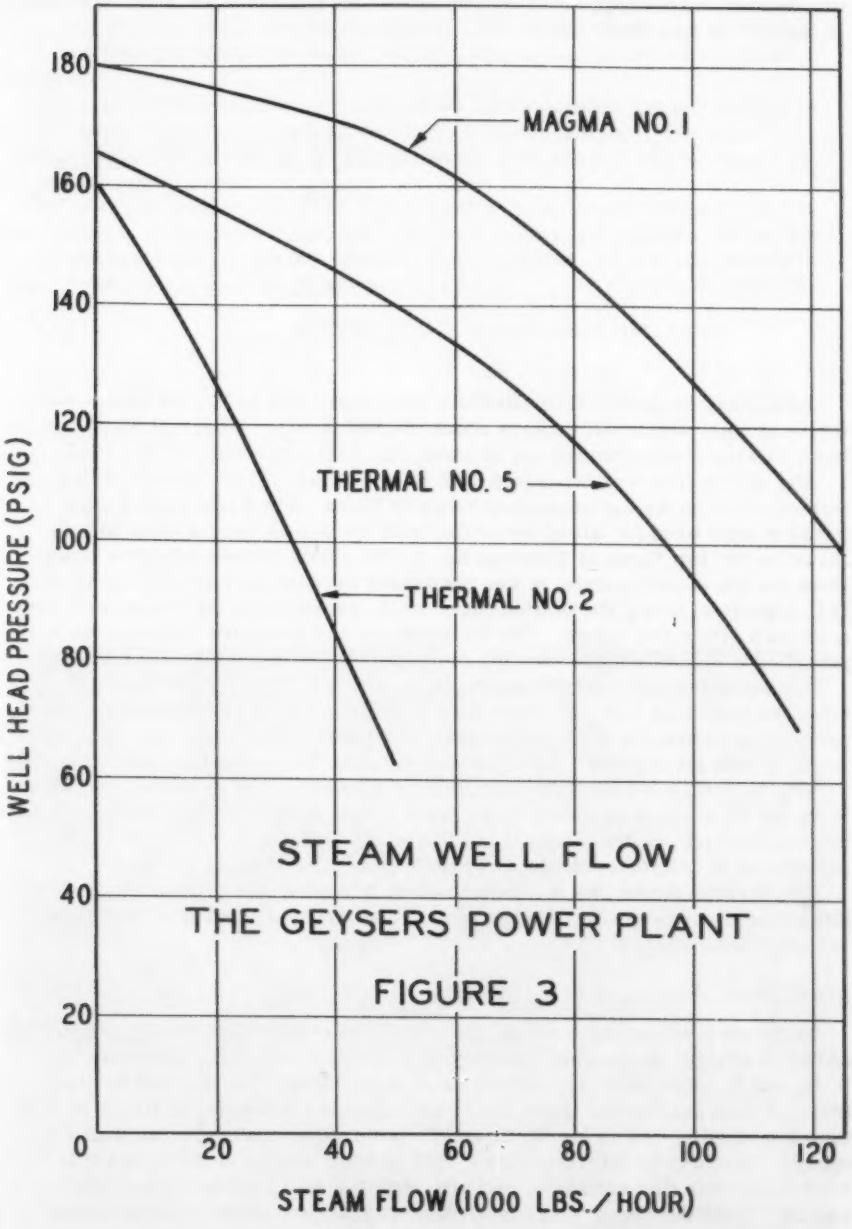

MAGMA NO. 1
THERMAL NO. 5
THERMAL NO. 2
WELL HEAD PRESSURE (PSIG)
STEAM FLOW (1000 LBS./HOUR)
0
20
40
60
80
100
120
140
160
180
STEAM WELL FLOW
THE GEYSERS POWER PLANT
FIGURE 3

Fig. 3.

square inch gage when the turbine inlet pressure is 100 psig. The measured steam flow and enthalpy (heat content of steam) for the three wells are shown in Table 1 at this condition.

TABLE 1

	115 Psig at Steam Wells	
	Steam Flow Lbs/Hr	Enthalpy Btu per Lb
Magma No. 1	111,000	1207
Thermal No. 5	82,000	1199
Thermal No. 2	26,000	1204
Total for three wells	219,000	

Additional wells will be drilled in order to provide sufficient steam for the first unit, which will require about 250,000 lbs/hr. Thermal No. 4 has been blowing steam into the air at about this rate since September, 1957.

The steam flow was measured with flow nozzles. Three steel nozzles were used to cover the necessary range of flows. The 6-inch and 4-inch nozzles were used for all of the wells, with the 3-inch nozzle used only to measure the low flows of Thermal No. 2. To obtain proper entrance conditions for the 6-inch nozzle, it was necessary to make an inlet pipe of 18-inch O.D. pipe (containing the thermometer well) connected to the steam well with a 12-inch elbow and flange. The temperature and pressure readings made it possible to determine the enthalpy since the steam is slightly superheated.

A view of the flow test arrangement is shown in Fig. 4. The steam blasts from the nozzle at high velocity with a terrific roar. It is necessary to work around the equipment with ear plugs or ear muffs of the type used by persons working with jet engines. All communication is done by writing notes.

The formula used for determining the steam flow is $W = 1080\ A \sqrt{P/V}$ W is the steam flow in lb/hr, A is nozzle throat area in square inches, P is the nozzle inlet pressure, and V is the specific volume in cubic feet/lb corresponding to the temperature and pressure of the steam at the inlet.

The various pressures were obtained by adjusting the valve at the wellhead. Simultaneous readings were taken of wellhead pressure and inlet nozzle pressure and temperature.

Non-Condensable Gases in the Steam

Tests were made of the steam for content of non-condensable gases in 1925, and in 1956 from Magma No. 1 before it was redrilled. The 1958 tests by P. G. and E. were made to confirm these tests but since they showed much less gas than the former tests, they were repeated a number of times to confirm the results. All of the P. G. and E. tests were made with the well fully opened. In order to determine the effect with the well shut down, several tests were made with this condition, and also with the well flowing only a small amount. Considerably greater quantities of gas were obtained under these conditions, indicating the importance of the well being open when testing to obtain results comparable to what may be expected when operating.

It is believed that the steam and gas come from a common source for all the wells as the quantities found in each well are approximately the same, although the earlier tests vary more than the 1958 tests. This may be because there was less flow from the wells or the testing technique may have been improved.

The tests of the steam from the three wells tested show the percentage of non-condensable gases to be from 0.68% to 0.83% by weight. The weighted average of the steam flow to the turbine would give about 0.75% by weight of non-condensable gases.

The greatest proportion of the gas is carbon dioxide with methane, hydrogen, nitrogen, hydrogen sulfide, and ammonia in smaller amounts. A weighted average of the gases in the steam to be supplied to the turbine is shown in Table 2. The average molecular weight is 34.37.

TABLE 2

Gas		Volume Percentage at 60 F and 30" Hg	Weight Percentage
Carbon Dioxide	CO_2	69.32	88.73
Methane	CH_4	11.81	5.49
Hydrogen	H_2	12.70	.74
Nitrogen, Argon	N_2 and A	1.59	1.29
Hydrogen Sulfide	H_2S	2.99	2.96
Ammonia	NH_3	1.59	.79
		100.00	100.00

FIG. 1 - STEAM FLOW TEST EQUIPMENT

Test for Wet-Bulb Temperature

In August 1958, a recording dewpoint and dry bulb instrument was installed at the approximate location of the cooling towers. An interesting feature of this installation was the small 1.5 KW steam turbine-driven 110 volt AC generator which was installed to furnish the heat required for the instrument. From the data obtained, it was determined that, although the maximum wet-bulb was 71°F, the high temperatures occurred such a small percentage of time the design wet-bulb for the cooling towers could be set at 65°F.

Corrosion Tests

It was realized early that the most complicating design feature of the plant would be to determine the most economical materials to use which would satisfactorily withstand the corrosion resulting from the gases in the steam. Reports of the corrosion from the steam in Italy and New Zealand were studied. These reports were helpful as a guide, but it was believed necessary that tests be made at The Geysers to determine effects of the gases there. Quick obtainability of test results was desirable also as specifications had to be written and equipment purchased to enable the plant design to proceed. It was therefore decided to construct a small scale model of the plant cycle from the turbine exhaust to include barometric condensers, gas ejectors and cooling tower, based on the use of a Corrosometer to measure the effects of corrosion on various materials.

The Corrosometer is an instrument which measures very small differences in amperage of a current passed through the test sample. The change in amperage is caused by the area of the hollow wire of the material being reduced by corrosion. The corrosion effects can be obtained in a week of exposure. The test probes for the instrument can be provided of the material to be tested either for pressure or an atmospheric condition.

The test set up was designed around the smallest standard barometric condenser available from stock. The steam flow was 310 lbs/hr, which gave a scale ratio of about one to 770 of the large scale plant.

In order to simulate the 11% moisture content of the turbine exhaust steam, a portion of the steam was run through a heat exchanger, cooled by creek water pumped to the storage tank located above the test stand.

Figs. 5, 6 and 7 show the equipment used for the tests. Fig. 5 shows the test stand with the steam inlet at the left and the gas exhausting from the after-condenser at the right. The test stand is located at Magma No. 1. The turbine-driven generator which furnished power to drive the cooling tower fan motors can be seen behind on the left.

Fig. 6 shows the hotwell which had to be located below the test stand to establish the barometric leg on the condenser. The small steam driven reciprocating water pump (shown to the left of the hotwell) pumps the circulating water to the cooling tower. Steam is obtained from Thermal No. 2 seen in the left of the picture.

Fig. 7 shows the two cooling towers with the 60,000 gallon water storage tank in the background. Each tower is rated to cool 15 GPM of 120 F water to 80 F with 65 F wet-bulb air.

The piping of the test stand was prepared to test the pressure probes in three places; first, the turbine exhaust duct; second, the pipe to the gas ejector; and third, the gas outlet from the after-condenser. The atmospheric type probe was used in the condenser hotwell.

Probes were obtained of carbon steel, and Types 410, 304 and 316 stainless steel. The material cost is in the same order as listed. It was desirable to obtain test information so the lowest priced material with adequate life could be specified.

The Corrosometer readings are obtained in a form which is readily evaluated in mils per year corrosion. The relative corrosion rates are shown in Table 3. It will be noted that carbon steel did not have an adequate life in any of the locations, whereas all types of stainless steel could be used.

TABLE 3

	Corrosion Rates in Mils per Year			
	Carbon Steel	Type 410 SS	Type 304 SS	Type 316 SS
Turbine Exhaust Duct	10.95	0.31	1.06	0
Pipe to Gas Ejector	49.6	1.25	1.06	0.58
Gas Outlet from After-Condenser	86	-	-	-
Condenser Hotwell	171.5	0	2.82	0

Aside from the determination of the materials to be used in the barometric condensers, gas ejectors, and associated piping, it was necessary to determine what materials are satisfactory for use in the circulating water pumps and piping. Samples of concrete, aluminum, wrought iron, and cast iron were therefore placed in the hotwell.

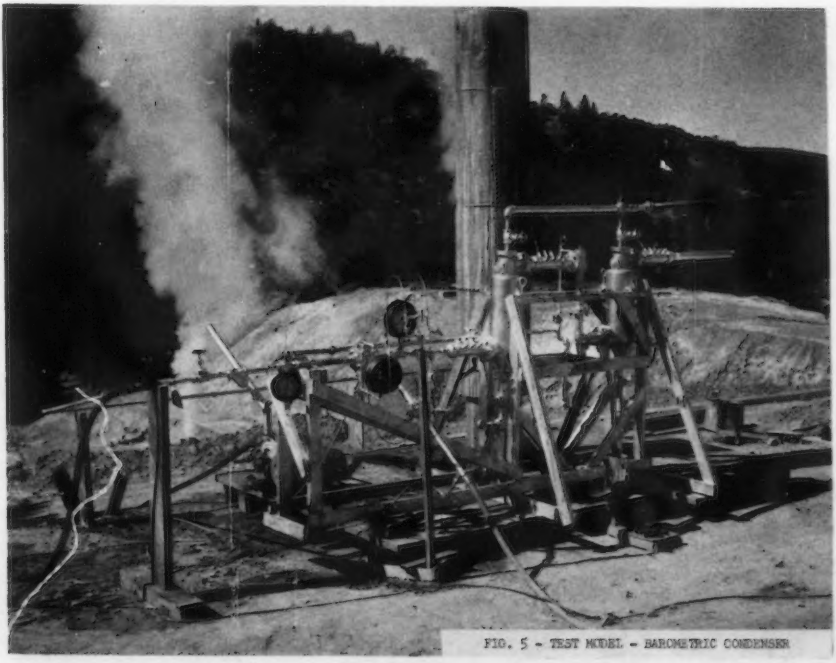


FIG. 5 - TEST MODEL - BAROMETRIC CONDENSER

To test the effect of the aeration and degasification in the cooling tower, the proves were then removed from the condenser hotwell and installed in the cooling tower basin. The test equipment was inspected three days after the proves were installed and it was found that the Type 410 SS probe had been practically destroyed. In order to confirm this phenomena, a test coupon of Type 410 SS was placed in the basin where it was discovered that some type of growth appeared on the sample. Neither the Type 304 SS nor Type 316 SS had the growth on them.

The results to the end of December 1958 were that Type 316 stainless steel showed no corrosion in any of the places tested. Type 410 stainless steel had rapid corrosion in the cooling tower basin, as did the carbon steel test probes. So far the aluminum test coupons look satisfactory.

Test coupons of Type 430 were later inserted in the cooling tower basin to determine if all chromium type stainless steels would be adversely affected. The early results show the same corrosion effects as for Type 410.

The tests of the concrete samples have not been completed but superficially they do not appear to be affected.

Tests for the Chemistry of the Cooling Water

Tests showed the water in the cooling tower basin to have a pH of 7.8, while the condenser hotwell water had a pH of 7.0. Theoretical calculations of the absorbability of the gases in the water in the main barometric condenser showed that relatively minor amounts of the gases would be absorbed except for ammonia which has a high rate of absorption. This is true at the

FIG. 6 - TEST MODEL - CONDENSER HOTWELL

4 inches Hg absolute pressure in the main barometric condenser. The inter-condenser would operate at 11-12 inches Hg absolute pressure so more gas would be absorbed in the water, and even more would be absorbed in the after-condenser operating at a pressure of one (1) lb per square inch gage. The flow of water to the latter two vessels is expected to be about 330 GPM and 200 GPM respectively with the flow to the main condenser being 11,000 GPM. Calculations showed that the total gases in the water should be about 35 parts per million for the first cycle. It was expected also that a considerable part of the gases would be stripped in the cooling tower.

Calculations also showed that the same amount of gases would be contained in the main condenser effluent when operating at 4.6 inches Hg as would be contained in the total effluent of all three condensers operating at the design conditions. Therefore, it was decided to not operate the test model's after-condenser and run the test model's main condenser at about 4.6 inches Hg. The steam jet gas ejector then was exhausted to the atmosphere.

The test model was operated until a stable condition was obtained and water tests taken. To ascertain what the conditions would be in the inter-condenser and after-condenser, readings were taken also at 11" Hg and atmosphere pressure.

The tests showed that a large part of the carbon dioxide was stripped and the hydrogen sulfide was changed to elemental sulphur in the cooling tower. However, the ammonia built up ti 81 ppm in the water leaving the cooling tower although it was calculated to be only 2.4 ppm in the first cycle. The effect of this is to make the circulating water have a slightly alkaline pH in the

FIG. 7. TEST MODEL - COOLING TOWERS

main barometric condenser. The effluent from the inter-condenser and the after-condenser, however, are expected to be slightly acidic. This is indicated by the tests made December 22-23, 1958, the results of which are shown in Table 4.

TABLE 4

PH of Circulating Water

Operating Pressure of Barometric Condenser	5" Hg Abs	11" Hg Abs	Atmospheric
PH of Water Entering at Top of Cooling Tower	7.21	6.59	6.60
PH of Water Leaving at Bottom of " "	7.65	7.62	7.82
PH of Water Leaving Barometric Condenser	7.20	6.60	6.41

There was some entrainment of gas in the cooling water leaving the condenser. In the full scale plant, it would be expected that the pH at this point would be higher.

Tests for Radioactivity of Steam Condensate

A sample of the steam condensate was tested with the result that the alpha radiation was found to be about 0.015×10^{-7} micro curies per milliliter and the beta and gamma radiation were no more than the background radiation. This is well below the permissible level established for drinking water by the National Bureau of Standards and the National Committee for Radiation and Measurement.

Geology of the Area

To the casual visitor, engineer or otherwise, the topographic features appear typical of the low level areas of the coastal mountain ranges of Northern California. Except for localized areas of surface discoloration, lack of vegetation, and escape of steam from the natural vents and man-made steam wells, there is little on the surface to differentiate the canyon of Big Sulphur Creek from several others in the area. That the geology at relatively shallow depth is significantly different from that at the other canyons is amply proven by the successful completion of a number of producing steam wells, some of which were drilled in 1922-1925, others in 1955 and 1957.

To guide the engineering and economic decisions which had to be made prior to P. G. and E.'s decision to build a power plant, a number of geological reports were made available for study. The geological information reported here is a generalized summary of the conclusions reached in these geologic reports which date back to 1924. Although there is good agreement on the question of longevity of the steam supply, there are various opinions as to its origin. A consulting geologist was also retained to advise on specific problems arising in the actual design of the plant at the particular site chosen.

The canyon of Big Sulphur Creek cuts through an area of low mountains; the elevation at the site of initial development being about 1500 feet. Most of this area is underlain by a thick sequence of sedimentary and volcanic rocks of the Franciscan formation which are intruded by tabular bodies of serpentine

of about the same age. The area where fumaroles occur is usually at or near the base of a large serpentine intrusion that crops out along the northern slopes of Big Sulphur Creek Canyon. This serpentine body delineates a thrust fault of large magnitude, and was possibly intruded along the plane of the thrust when it was active. The presence of this impervious body of serpentine (acting as a ceiling) and numerous cross faults of the rocks below possibly provide the collection system and channelways necessary for the concentration and escape of steam from deep within the earth at this particular location, although no actual confirmation of this has been made.

Origin of the Steam

While the above is believed to be a fairly correct, though brief, description of the geologic structure of the site and offers a possible explanation of why the steam emerges where it does, it does not explain the origin of the steam.

Two principal theories have been offered to explain the origin of the steam, both presuming that the primary source of the necessary heat should be considered a body of rock that has been heated by large scale magmatic or metamorphic processes. The two theories differ as to the source of water from which the steam is produced; one theory being that surface water (meteoric) has somehow found its way down to the heat source; the other theory holding that the steam is being distilled from magma, in the process of solidifying, or granite which has been sufficiently heated to throw off its water of crystallization. Another possibility is that both of the above sources are operative and that the steam is produced from a mixture of magmatic and surface waters. This seems to be a reasonable assumption, although it is not possible to divide percentages of total output between the sources.

Longevity of the Supply of Steam

Regardless of where and how the steam originates, the problem of whether of not the supply is sufficient to justify the installation of a generating plant remains the paramount question. There seems to be little doubt that the heat supply is sufficient for a period of time longer than necessary for consideration from an economic viewpoint. However, the extent of the supply of water from which steam can be produced is not known. There is little likelihood of exhausting the supply of the steam actually produced from the magma.

However, some question arises as to how much water can make its way down from the surface to the heat source to be turned into steam and returned to the surface for use. The first wells drilled in 1922-25 are reported to be producing today at the same volume as when first drilled. The wells (new or old) show only minor interaction of pressure or steam flow, even though they are relatively close together, which is perhaps indicative of the large size of the available steam supply.

Fumaroles in the area have been known to exist for over 100 years, and hearsay is that the activity has been more or less constant. Areas of similar activity throughout the world have been known to be in existence for hundreds of years, particularly the areas of Italian development.

Description of the Plant Cycle

Before discussing the arrangement details, the plant cycle will be outlined. The principal differences between the usual modern power plant and a geothermal power plant are (1) there is no boiler and, therefore, steam condensate is not recycled but is used for cooling water make-up, (2) the steam pressures are lower, and (3) the steam has much larger quantities of non-condensable gases which must be removed from the condenser to maintain the vacuum.

A characteristic of the flow from a steam well is that the pressure drops with an increase in flow. Therefore there is a point at which the most power can be obtained with a combination of flow and pressure. It was decided for the first unit that a pressure of 100 lbs per square inch gage would be used.

As it is not necessary to return the steam condensate, it was decided to use the less expensive barometric condenser wherein the condensing steam and cooling water mix. The mixture falls down a barometric leg into a hot-well and is then pumped to a cooling tower where the mixture is cooled by evaporation.

A very important feature of the cooling cycle is that a lesser amount of water is lost by evaporation than is supplied by the condensed steam, which means that no external make-up is required except to fill the cooling system initially. The amount that will overflow from the cooling tower is 12% to 40% of the steam condensate depending on the atmospheric conditions. The greater amount is obtained during the cooler periods of the day or season.

The non-condensable gases in the steam require a much greater capacity gas ejecting equipment than conventional units. Because of its lesser cost, it was decided to use steam jet equipment instead of mechanical gas pumps.

The steam, as it comes from the well, contains solids which, if they were not removed, would cause additional wear to the turbine blades. As a part of the contract, the steam suppliers are required to remove all the particulate matter that can be separated by the best commercial equipment available.

The flow diagram for the cycle is shown in Fig. 8. The unit is supplied with 240,000 lbs per hour of 100 lbs per square inch and 348 degree Fahrenheit steam. The first stage jets will require 5,300 lbs of steam per hour and the second stage jets will require 4,900 lbs per hour. The main barometric condenser requires 11,000 gallons per minute of 80 degree F cooling water and the inter-condenser will use 310 GPM and the after-condenser 200 GPM. An economic study was made of the best vacuum to use for the condenser which for this unit was decided to be 4 inches Hg absolute pressure. A better vacuum would require additional cooling water, more gas ejector steam, larger circulating water pumps, and a larger cooling tower. For future units, the study may show that 3 inches Hg absolute is less expensive. There were circumstances connected with the use of the existing turbine for the first unit that made 4 inches preferable.

The only other cooling water requirement for the unit is that of the turbine-generator oil cooler which will require 90 GPM. The generator will be air cooled, using filtered air from outdoors.

Civil Engineering Features of the Plant

Access to the Site

A paved road off of U. S. Highway 101 above Healdsburg extends to "The Geysers" resort. At present the power plant site is reached from the resort

FLOW DIAGRAM
THE GEYSERS POWER PLANT
FIGURE 8

over an unimproved dirt road along the south bank of the Big Sulphur Creek, and then over a ford about one-half mile east of the resort. A permanent bridge will be constructed to provide all weather access to the plant since Big Sulphur Creek at times carries sufficient water to make fording impossible.

Water Storage and Fire Protection

To insure a supply of water under gravity pressure for fighting fires, it is planned to install a 50,000 gallon storage tank on the hillside to the north of the plant. This tank will also serve as a storage tank for general service water, pumped from the creek. Under ordinary operating conditions, no make-up water will be required for the condenser cooling water system, as the steam cycle, mentioned before, has an excess of condensate. Domestic water will also be supplied from this tank, although this will be a very small amount since it is planned that the plant will be unattended under normal conditions.

Condensing Water Cooling Towers

The cooling towers will be located adjacent to the creek and 5 ft. below the plant elevation. They will provide cooled water by gravity flow through 27 inch reinforced concrete pipe to the coldwell sump for the pump which supplies water to the condenser. After falling through the barometric condenser and condensing the steam, the water will be picked up from the hotwell and pumped back to the cooling tower to be cooled and recycled.

The cooling towers will be of standard induced draft design having three cells. Special attention has been given to offset the corrosive effects of the water. Plastic fill and stainless steel hardware will be used. The cast iron column anchors will be Heresite coated. The basin of the cooled towers will be of reinforced concrete with a coal tar coating.

Due to the cooling duty inherent in the cycle, the cooling towers will be operated with a continuous blowdown of 12% to 40% of the condensed steam which provides the make-up. This is expected to limit the buildup of undesirable chemicals in the cooling water to acceptable limits. The problem of disposing of the blowdown flow has not been finally resolved. If the quantity and chemical contaminants are such that direct discharge to the creek is not satisfactory, suitable means of retention, and treatment if required, will be provided.

Switchyard Facilities

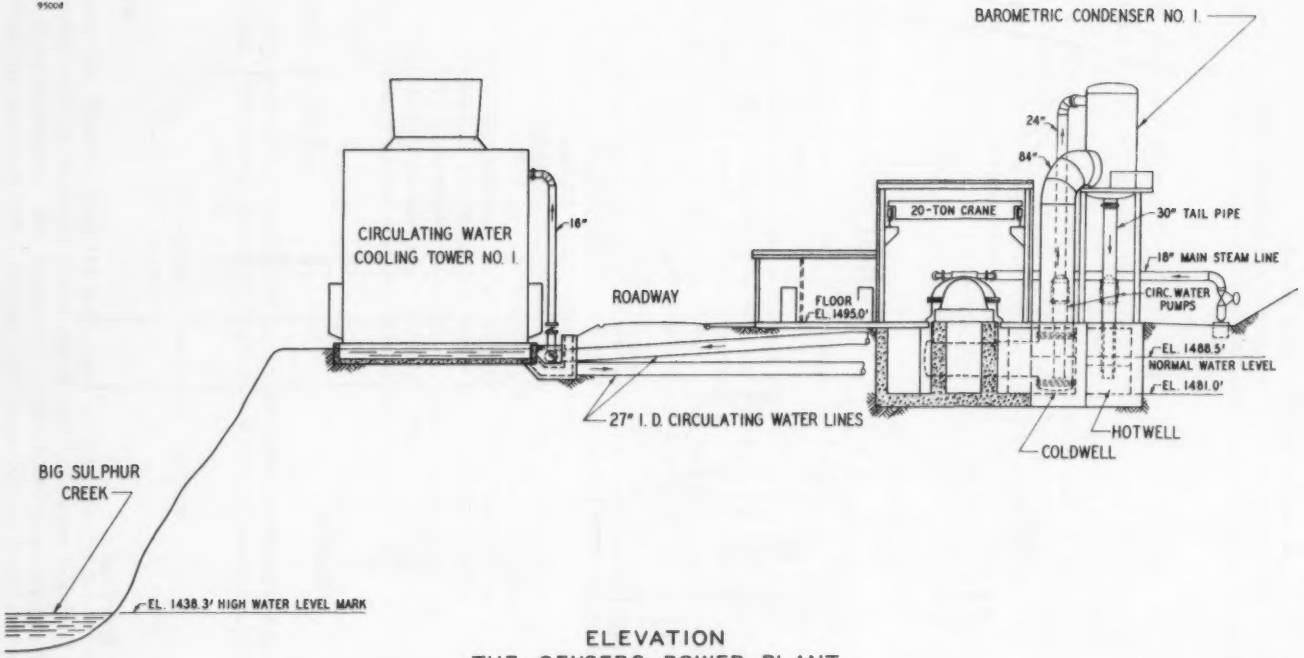
The switchyard will contain the necessary step-up transformers and switchgear. The major switching structure will be of aluminum construction.

Structural Features of the Plant

General

Due to the elimination of the boiler and a large part of the accessory equipment found in the usual steam plant, the structures required are required to be minimum. The simplicity of the layout is shown in Figs. 9 and 10. While for the first unit a shelter will be provided, future units may be

PLAN
THE GEYSERS POWER PLANT
FIGURE 9



ELEVATION
THE GEYSERS POWER PLANT
FIGURE 10

installed entirely out-of-doors. This description must be considered as a preliminary one as some of the details have not been determined yet.

Foundation Conditions at the Plant Site

The plant will be located on a relatively flat area on the north bank of Big Sulphur Creek at an elevation about 65 feet above the creek bed. The bank of the creek is very steep at this point, approximating a 1 to 1 slope. Because of the unique nature of the geology of the area, it was decided to undertake a somewhat more extensive soils investigation than would ordinarily be required to develop adequate foundation design criteria and to prove the overall stability of the site.

Seven test holes were drilled to explore the subsurface conditions in the area required for Unit No. 1 and future Unit No. 2. Two of these holes were deep (scheduled to 75' below surface) and were located under the barometric condensers. The remaining five holes were 15' to 25' in depth and were scattered at the boundaries of the site to confirm the extent of the subsurface conditions found at the deep holes.

The borings generally showed that the site is underlain with layers of decomposed sandstone and serpentine which will provide adequate bearing capacity.

An interesting subsurface condition is the existence of high ground temperatures at shallow depth below the surface. At a depth of about 55 feet below the surface, the soil temperature was measured at 170°F in the deep test holes. This had been anticipated and equipment to measure the ground temperatures provided.

The possible chemical reactivity of the soil and ground water with the concrete is being considered. Evidence of hydro thermal alteration of the sandstone and serpentine was noted in the test samples.

Turbine Building

The turbine building will be of rigid steel frame design, 31 feet by 77 feet in plan with an eave height of about 30 feet. Space also will be provided alongside of the turbine building for an office change room, maintenance shop, and storeroom. At the east end of the turbine room, space will be provided for electrical switchgear, the exciter, and battery. The exterior sheathing for the building will be aluminum in order to resist the corrosive atmosphere. The turbine building will be provided with a 20 ton traveling crane to assist in the initial erection and maintenance of the unit.

Turbine-Generator Foundation

The turbine-generator operating level will be at grade, primarily to reduce the overall height of the building. This will be a unique feature of the plant although it necessitates a partial basement. The turbine-generator foundation will be reinforced concrete with the basement walls forming the substructure. The conventional water cooled surface condenser usually located just below the turbine will be replaced by an exhaust tee and 84-inch diameter duct which will carry the exhaust steam to the barometric condenser, that will be supported 28 feet above the operating floor level on a structural steel tower.

Hotwell

The hotwell will consist of a reinforced concrete pit. The purpose of the hotwell is to permit the installation of a 34-foot long, 30-inch diameter barometric tail pipe on the bottom of the condenser. The tail pipe will have a two foot seal which will be maintained by a weir between the tail pipe pit and the circulating water pump pit. The water will flow over the weir and will be pumped to the cooling tower. The flow of circulating water will be about 12,100 GPM.

Mechanical Features of the Plant

Turbine

The turbine for the plant is to be designed for 100 lbs per square inch and 348 degrees F inlet steam conditions and a back pressure of 4 inches Hg absolute. There was available a 12,500 KW turbine-generator unit which was installed in 1924 as Unit No. 2, in the P. G. and E.'s Sacramento Power Plant. This turbine is a General Electric Company, 9-stage, 1800 rpm, single casing, condensing turbine, designed for inlet steam conditions of 175 psig with 100 degrees F superheat and 1-1/2 inches Hg absolute exhaust pressure.

The turbine will be modified by opening up the steam passages. The control valves will be replaced by two butterfly valves, 6" and 10" and the first three stages will be removed. The remaining six diaphragms will be replaced because it is believed the copper brazing, used in the original construction would be deteriorated by the hydrogen sulfide in the steam. The 8th and 9th stage buckets will be changed to increase the flow through them, and will be constructed of 11-13% chrome steel. The diaphragms of the 8th and 9th stages will also be enlarged to permit more steam flow. The monel buckets now installed in the 4th to 7th stages will be retained as no mercury has been found in the steam, although there are mercury mines several miles away.

It is expected that there will be fine mineral matter in the steam, which might cause the stem of the turbine throttle valve to stick. For this reason, an emergency stop valve of the swing check type will be installed after the throttle valve. Simultaneous tripping of the two valves will insure positive stopping of the steam flow.

Barometric Condenser

The use of a condensing type unit will provide about twice the electrical output per pound of steam compared with one which exhausts to the atmosphere, and therefore it was agreed that a condensing type turbine would be used. As was explained before the barometric condenser was selected because it is not necessary to conserve the steam condensate. In fact, the mixing of the steam with the cooling water in the condenser is a simple way of providing the make-up requirements. Also, it makes possible a less expensive means of degassing the condensate than could be accomplished in a surface condenser.

The condenser vessel will be 120 inches in diameter by about 23 feet long, mounted vertically, with an 84-inch steam inlet. The cooling water inlet connection will be 24 inches, and the outlet 30 inches. The first stage ejector will be mounted on the vessel with the gas connection at the top of the vessel.

The mixture of gases and motive steam will be injected into the inter-condenser. The cooling water will condense the steam and the mixture will drop through a tail-pipe into the hotwell. The second stage gas ejector will be mounted on the inter-condenser. Its mixture of gases and motive steam will be injected into the after-condenser, and the cooling water from the after-condenser with the condensed steam will be drained into the hotwell. The inter-condenser will operate at a pressure of 12 inches Hg absolute and the after-condenser at a pressure of one (1) lb per square inch gage.

Instead of exhausting the gas and steam mixture from the second stage gas ejector into the atmosphere, an after-condenser will be provided so the amount of contaminated moisture ejected will be reduced. The gases exhausted from the after-condenser will have a minor amount of vapor which, except on very cold days, will not condense and fall on equipment in the vicinity.

The condensers will be fabricated from carbon steel clad on the inside with Type 316 low carbon stainless steel. The gas ejectors and internal parts of the condensers will be fabricated of solid 316 stainless steel.

Circulating Water Pumps

The circulating water pumps will be vertical turbine type. The condenser supply pump, one full size, taking suction from the coldwell will have a capacity of 11,600 GPM at 29 ft. head. The full size cooling tower supply pump taking suction from the hotwell will have a capacity of 12,300 GPM at 50 ft. head. All parts in contact with the water will be made of Type 316 stainless steel.

Cooling Towers

The cooling towers will be of the induced draft type designed to cool 12,000 gallons per minute of water from 120 degrees to 80 degrees Fahrenheit with a wet-bulb of 65 degrees F.

Turbine Lubricating Oil System

The turbine oil cooler obtained with the machine will have the admiralty tubes and bronze tube sheets replaced with aluminum in order to withstand the corrosion of the cooling water.

The Bowser continuous filter and oil storage tanks provided with the unit in 1924 are in good condition and will be used. The turbine oil overflows from the reservoir in the turbine to the filter located in the basement. It passes through the filter and is elevated to the reservoir by the 4 GPM lube oil filter pump.

Service Water Make-up Pump

The only other principal mechanical auxiliary is the service water make-up pump which will pump water from the creek to the 50,000 gallon water storage tank on the hill. It will be designed for 100 GPM and 300 foot head, and operate automatically, controlled by float switches on the tank.

Electrical Features of the Plant

General

The main electrical components of the plant will consist of the generator,

generator circuit breaker, main step-up transformer, 60 kv transmission line oil circuit breaker, a unit auxiliary transformer and 480 volt switchgear, along with motors and control devices.

Generator

The alternating current generator procured with the 1924 unit is rated 12,500 kva, unity power factor, 3 phase, 60 cycles, and 11,500 volts. It will be air cooled by taking air through dry-type filters located outside the building, and exhausting the heated air in a duct through the roof. A removable section will be provided in the duct to pass the crane. The generator will be grounded through a distribution transformer and secondary resistor.

The exciter for the generator will be motor-driven and was originally located at the Sacramento Power Plant. It is designed for 100 KW, 125 volts, 800 amperes.

Generator Circuit Breaker

The generator circuit breaker which will connect the generator to the stepup transformer will be an indoor metal-clad air circuit breaker rated 13.8 kv, 1200 amps and will have a 250 MVA interrupting rating. Required potential and current transformers for metering, relaying and control will be located in the breaker housing.

Main Step-up Transformer

The main step-up transformer will be rated 10,000/12,500 KVA, OA/FA, 11.5 kv/60 kv, 3 phase, 60 cycles. The 11.5 kv winding will be wye connected, and the 60 kv winding will be solidly grounded wye connected.

The step-up transformer will be located in the switchyard at the east end of the plant.

Transmission Line Oil Circuit Breaker

The transmission line oil circuit breaker which will connect the high voltage winding of the main step-up transformer to the 60 kv transmission line will be rated 69 kv, 600 amps and will have 1000 MVA interrupting rating. This OCB will be located in the plant switchyard.

Plant Auxiliary System

The plant auxiliaries will be fed from a 650 MVA, 11.5 kv/480 V delta-delta 3 phase, 60 cycle auxiliary transformer, located just outside the north wall of the plant building near the northeast corner. The high side of this transformer is tapped in between the generator air circuit breaker and the main step-up transformer. This will permit feeding the plant auxiliaries during plant startup (with the generator breaker open) from the 60 kv system.

The various plant auxiliaries will be controlled by combination motor starters.

60 kv Transmission

The electrical energy produced at The Geysers Power Plant will be fed into the P. G. and E. system over a new 60 kv transmission tap which will tie into the existing Fulton-Hopland 60 kv circuits approximately 10 miles from

the plant. This new 60 kv tap with a capacity of 34,000 KVA will be adequate for another comparable unit at this site. Any additional units will tie into P. G. and E.'s 110 kv system and will be located at another site.

Automatic Features of the Plant

The plant is being designed so it can be operated automatically without attendance. When an operating condition occurs that will endanger the unit, it will be shut down and an alarm signal sent to the attended Fulton Substation. Other abnormal operating conditions that will not endanger the unit immediately will be transmitted by another alarm signal to Fulton Substation and will alert the operator so that a trouble man in the area can be dispatched to the plant to investigate.

An operating condition that is serious enough to trip the unit requires investigation at the plant before the unit is re-started. For this reason and because of the much greater cost of control equipment to automatically start a unit, manual control of starting is planned.

It is also planned to install automatic synchronization equipment so the generator can be re-synchronized with the system after a line fault has cleared. When a line fault trips the oil circuit breaker connecting the 11.5/60 kv transformer to the line, the unit will automatically govern itself to carry the 500 KW auxiliary load. After the line fault is cleared, the unit will be automatically reconnected to the line and loaded to 12,500 KW. The re-synchronization equipment will be designed so it can be used to operate the air circuit breaker to synchronize the unit to the system.

SUMMARY

The Geysers, discovered in 1847, have been developed to provide sufficient steam to operate a 12,500 KW unit. The steam condensed in a barometric condenser will provide sufficient make-up water for the cooling tower evaporation and spray losses plus 10 to 25% additional for blowdown. The unit will be designed to operate automatically with remote alarm to the attended Fulton Substation.

Pacific Gas and Electric Company's engineers are preparing the designs and its Construction Department will supervise construction of the plant which is planned to steam initially about April 1, 1960 with construction work to start in August 1959.

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

THE STRATTON STEAM ELECTRIC STATION

C. H. Alban,[1] F. ASCE

SUMMARY

Planning and construction of the first four units of the Ohio Edison Company's 700 megawatt Stratton Station scheduled for completion in 1962 will extend over a decade.

Simultaneous construction of the Corps of Engineers New Cumberland Locks and Dams and of the State of Ohio's improvement to Route 7 required much joint planning and coordination to utilize the narrow river valley at this locale.

Following World War II, consideration was given by the Ohio Edison Company to the Stratton, Ohio site on the Ohio River as a possible location for a large base load steam generating station. System planning considerations led to the prior development of another site near Niles, Ohio, before final plans were begun in 1950 to build on the Stratton site. Location of the principal power supply facilities is shown by Fig. 1.

During this same period, the Pittsburgh District of the Corps of Engineers was advancing its plans for a new dam and locks on the Ohio River immediately adjacent to the downstream end of the Stratton site. This development is the New Cumberland locks and dam.

In comparing plans, it was found that coordination of the navigation and power plant projects would be advantageous in the utilization of the land in the narrow river valley. In like manner, grading and drainage for State Route 7 relocation by the Highway Department was affected by the Corps of Engineer's design and contract. These problems were ironed out over the ensuing period of years by many conferences. Considerable "give and take" arrangements were necessary in order that the land available could adequately serve all projects.

Note: Discussion open until May 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2288 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 6, December, 1959.

1. Structural Eng. Commonwealth Associates Inc. Jackson, Mich.

Lake Erie
Mich.
Ohio
Pa.
W. Va.
Ohio
PRINCIPAL POWER SUPPLY FACILITIES
OHIO EDISON SYSTEM
COMMUNITIES SERVED RETAIL
COMMUNITIES SERVED WHOLESALE
FUEL PLANTS
345,000 VOLT LINES UNDER CONSTRUCTION
138,000 VOLT TRANSMISSION LINES
23,000, 34,500, 69,000 VOLT TRANSMISSION LINES
INTERCONNECTIONS
N
SCALE OF MILES
MARCH 12, 1959
THE TOLEDO EDISON CO
THE CLEVELAND ELECTRIC ILLUMINATING CO
OHIO POWER CO
OHIO POWER CO
THE DAYTON POWER AND LIGHT CO
COLUMBUS AND SOUTHERN OHIO ELECTRIC CO
DUQUESNE LIGHT CO
TORONTO
R. E. BURGER
STRATTON
Under Construction

FIGURE 1

The first step in the power plant development was to install the intake and discharge conduits across the new right of way for relocated highway, Ohio State Route 7.

This was accomplished by arranging with the Corps of Engineers to contract this work section of the job with the relocated highway grading which they were providing for the Ohio State Highway Department in consideration for the Government's taking over the land occupied by the original road.

The Corps of Engineer's contractor for the locks job made arrangements to use a portion of the power plant site for his construction site.

A brief schedule for the power plant site development follows:

1952-1954 - Tripartite negotiations for land utilization

1955 - Contract awarded for circulating water conduits under relocated highway; contract awarded for river intake and discharge circulating water conduits to powerhouse.

1956 - Contract awarded for service (maintenance) building and screen house for Units 1 and 2; contract awarded for main powerhouse, screen house extension for Units 3 and 4 and coal handling.

Unit 1 is scheduled to be in operation in the summer of 1959 with an additional unit scheduled to follow in the summers of 1960, 1961 and 1962.

Site

The site shown by Fig. 2 is at about Navigation Mile 54 on the Ohio River which is approximately 12 miles upstream from Steubenville, Ohio, and about 3 miles upstream from the Ohio Edison Company's Toronto Station. This particular site was chosen because of ample cooling water and plentiful and economical coal supplies. Land utilization is about as follows: 76 acres - station site; 13 acres - highway right of way; 20 acres - (Corps of Engineers) flowage easement; 340 acres of hillside land which will be used for transmission egress and ash disposal.

The Corps of Engineers disposed of approximately one million cubic yards of excavation from their locks job which with the spoil from the structures on the site raised the ground surface approximately 15'. This new elevation puts the yard grade well above any anticipated flood waters of the Ohio River.

The construction of the New Cumberland Dam provided a favorable pool condition which may be beneficial from the standpoint of circulating water supply.

Description

The present extimated cost of the completed 4-unit generating plant is approximately $118 million including site development and connections to the step-up transformers, but not including the outdoor transmission substation.

Each of the 4 turbine-generators have a guaranteed gross output of 185,000 kw at 1-1/2" back pressure. Their rating at 3-1/2" back pressure is 175,000 kw. They will be 3600 rpm tandem compound, triple flow type machine designed for 2400 psig, 1050 F initial and 1000 F reheat steam conditions. The annual heat rate based on a 75 per cent capacity factor has been calculated at 9150 Btu per net kwh.

FIGURE 2

SITE PLAN

Steam generators have a guaranteed output of 1,375,000 lb of steam per hour.

Each steam generator has 2 forced draft and 2 induced draft fans of airfoil design. Electrostatic precipitators are provided for fly ash collection.

Transmission will be at 138 kv for connecting the first 2 units to the system. Units 3 and 4 will tie into the new 345 kv lines being constructed to serve the Ohio Edison Company's central load area.

Fig. 3 - Plan View Operating Floor, and Fig. 4 - Cross Section, show the general arrangement of the principal equipment.

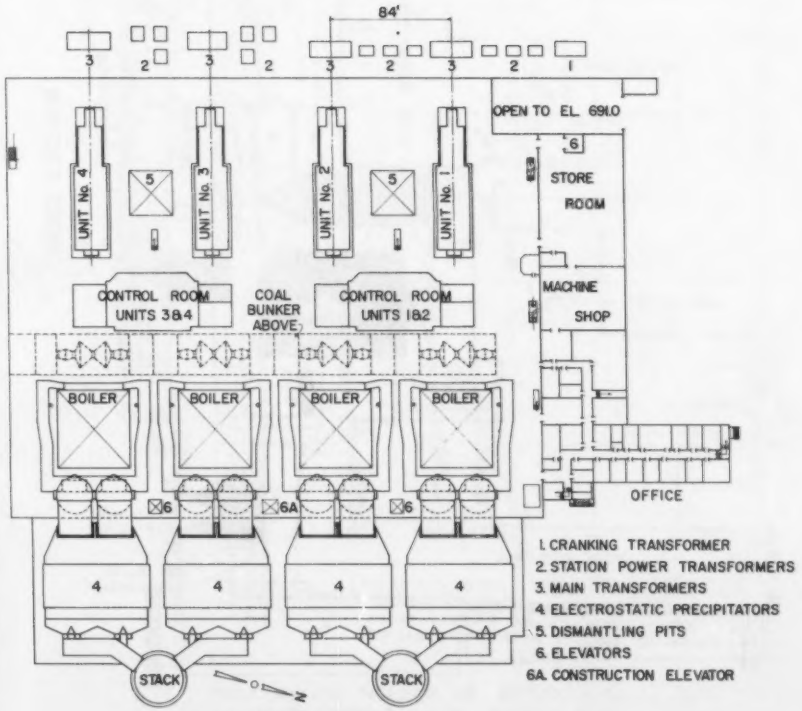
Demineralizer equipment is provided to supply make-up water.

Two combination passenger and freight elevators are provided for the operating plant employees. A temporary elevator was installed for use of construction personnel.

Construction work is on a continuing basis. The 4-unit power-house substructure was built as a single unit of work whereas the superstructure is being built in 2 phases. The first phase included Units 1 and 2 and the secong phase will include the superstructure for Units 3 and 4.

The substations, screen house and coal handling facilities were divided into 2 phases with distinct breaks in the continuity of the construction work.

The present construction schedule contemplates the completion of the 4-unit project in 1962.



PLAN VIEW
OPERATING FLOOR

FIGURE 3

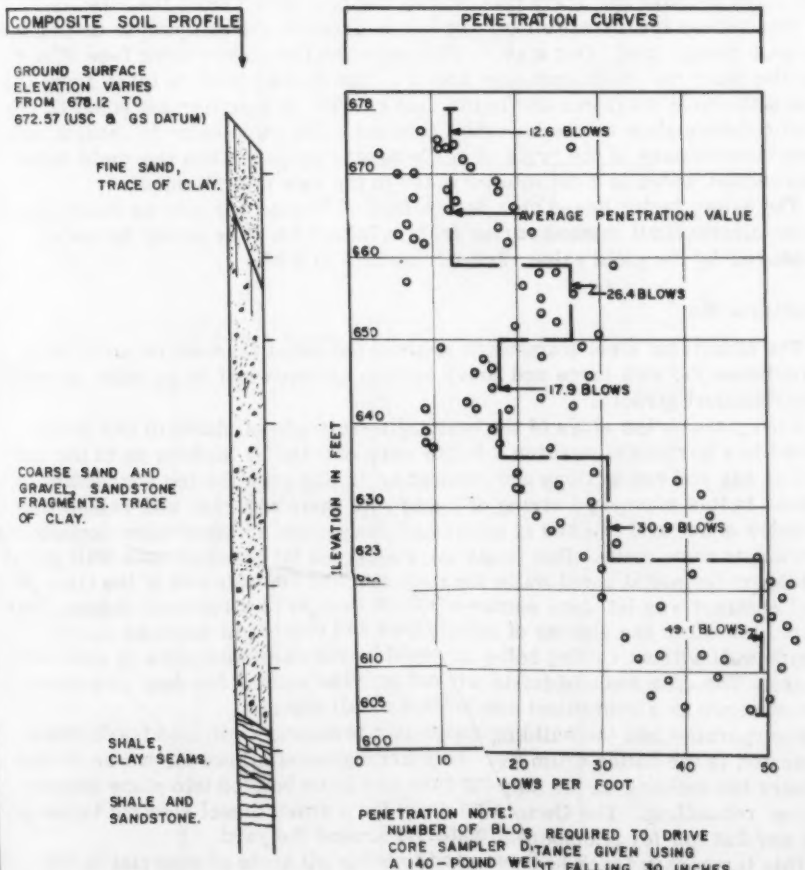
CROSS SECTION

FIGURE 4

Foundation Conditions

The soil below the previously mentioned fill is of river deposited material consisting of clay, gravel, fine sand, broken sandstone and shale to a depth of about 65', at which location a layer of solid sandstone is encountered. This overburden is not uniformly consolidated and, consequently, its supporting value is not consistently reliable. Penetration curves and a composite soil profile are indicated by Fig. 5.

Consideration was given to several alternate foundation designs, but it finally narrowed down to a choice between cast-in-place concrete piles and open-end pipe piles. For economic reasons, as well as the uncertainty and difficulty of obtaining delivery of steel pipe at that time, cast-in-place concrete piles were utilized.



PENETRATION VALUES OF BORING 1 THRU 7
STRATTON STATION, OHIO EDISON COMPANY
STRATTON, OHIO

FIGURE 5

A test program at the powerhouse location was set up to determine the supporting capacity of piles driven. Five piles were driven to various depths and driving resistances and were load tested to determine their ultimate load carrying capacity. The results of the tests indicated that should the piles be driven with an 80-C hammer (24,500 ft-lb energy) to 120 blows for the last foot of penetration, they would safely carry a design load of 75 tons per pile. Such driving would in most cases place the tip of the pile on or very close to the underlying sandstone layer.

There are approximately 7,000 piles under the powerhouse, chimneys, coal handling structures, service building and miscellaneous structures. Most of these piles have been driven to the 75 ton load carrying capacity. The few exceptions are for miscellaneous isolated structures, where a lesser load carrying capacity was required or where it was found desirable to utilize steel "H" piles because the heavy pile driving rig had moved from the site.

Two values for ultimate load were considered in the analysis to determine the safe design load. One may be designated as the elastic limit (see Fig. 6) and the other the yield limit (see Fig. 7). The elastic limit is taken from the load settlement diagrams and is the load carried at a settlement equal to the elastic deformation as measured by rebound. The yield value is determined from observations of the "rate of settlement" curves. When the yield value is exceeded, there is a definite increase in the rate of settlement.

The safety factor based on a design load of 75 tons per pile as determined by the elastic limit method varies from 1.72 to 5.53. The safety factor as measured by the yield value varies from 1.75 to 4.93.

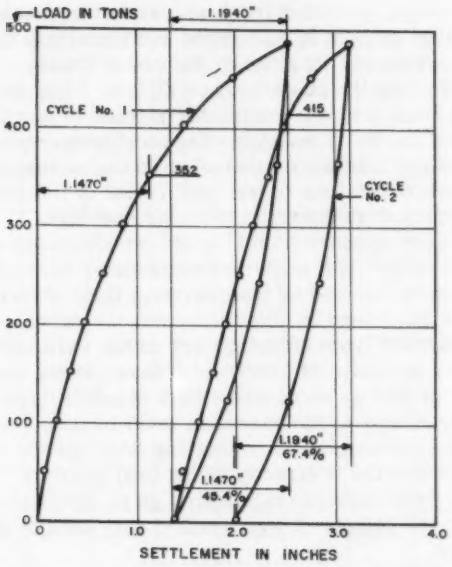
Structural Steel

The structural steel framework is about the usual framework for a large powerhouse job with large and heavy column sections and large built-up welded boiler support girders.

Wind girts on the sides of the building were made of standard bar joists placed in a horizontal position. Joists were erected by hooking on to the top joist at sag rod connections and connecting on sag rods for the succeeding joists. In this manner, a string of joists with their sag rods was raised like a window shade and erected at one time. Rods were adjusted when connections to columns were made. Bar joists were selected for the long span wall girts to support the metal panel walls because they are cheaper and at the time the steel contract was let were easier to obtain than were structural shapes. Bar joists also allow the placing of small pipes and electrical conduits up the outside wall without cutting holes as would be the case with girts of structural shapes. The open webs of joists will not provide a place for dust to accumulate or serve as a convenient catchall for small objects.

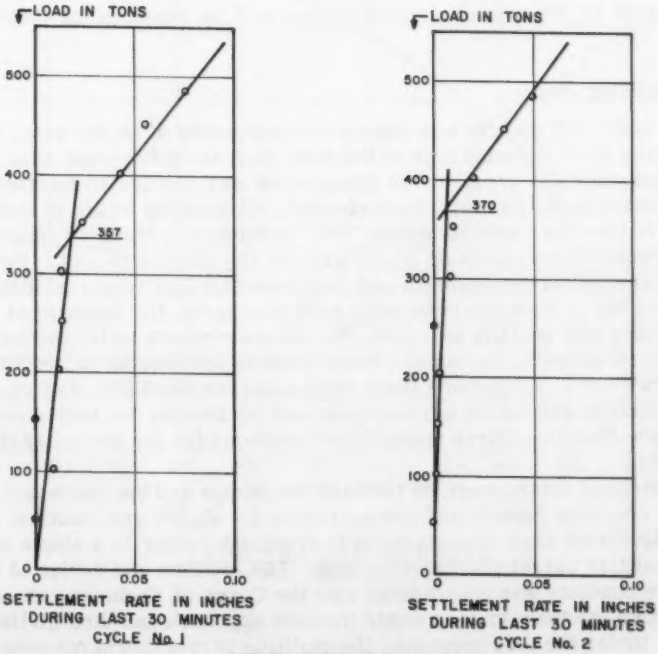
Incorporated into the building design is a temporary railroad track which is located in the boiler drum bay. This arrangement allows the boiler drums to enter the building on the shipping cars and to be hoisted into place without further rehandling. The Owner also provided a small diesel electric locomotive and flat car for transporting material around the yard.

This temporary track has been used to bring all kinds of material in the boiler room side of the plant and has contributed greatly to the speed of erection. Construction materials for the turbine room side are routed into the plant by the permanent track located so as to be available to the 200 ton turbine room crane. Temporary openings in the extension end of the plant,



LOAD-SETTLEMENT DIAGRAM
TEST PILE No. 3

FIGURE 6



YIELD VALUE CURVES
TEST PILE No. 3

NOTE: Time of load 60 minutes

FIGURE 7

together with ramps, provided truck access to the basement floor level. A surprisingly large amount of equipment and materials has been transported directly to its permanent location by the use of trucks.

Instead of erecting the powerhouse with 2 or 3 guy derricks as is quite often done, the steel erection contractor elected to use 2 crawler cranes with 140' booms and 30' jibs. After the steel was erected to the operating floor level, these cranes were ramped up to that elevation where he erected the steel in vertical sections to the roof. Most of the steel for the boiler room side was picked up through the boiler openings.

The cranes were supported on 8' x 18' wooden mats which rested on diaphragmed pairs of 24'' WF x 120 lb beams under each crawler tread. These runway beams were carried by the operating floor structural steel which in some cases was increased in size to support the erection loads.

Steel was unloaded from shipping cars in the yard and sorted on wooden pallets in proper sequence of erection. These pallets and their loads of steel were then picked up as required by 2 straddle-type carriers and delivered down the ramps to the basement level usually under the boiler openings. The crawler cranes on the operating floor picked the steel up from this location and erected it directly in its final position.

The straddle-type carriers eliminated extra handling of steel, since once it was placed on the pallets, it was handled only once to place in position for the erection rigs.

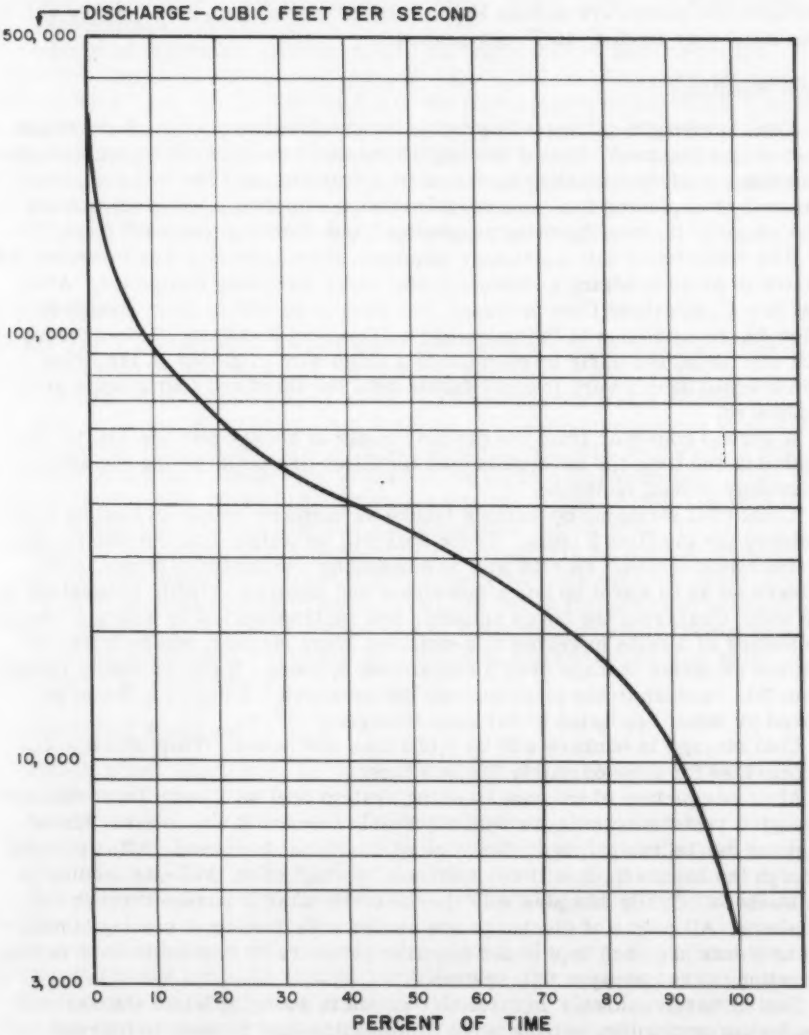
All principal field connections were made with high strength bolts. Bolts were tightened by torque controlled impact wrenches which were adjusted daily against a calibrator. Most of the bolts were inspected by an independent inspection agency.

About 12,000 tons of structural steel will be required for the 4-unit building.

Circulating Water

A limit of 2,000 cfs was placed on the quantity of intake water that could be taken from the bank side of the pool lying along the plant site. Any quantity substantially greater than this amount was deemed an interference with navigation in the lock approach channel. Circulating water is discharged back to the river approximately 1800' downstream from the intake and riverward of the upstream guard wall for the riverward lock. Because of the large initial development and the knowledge that future additions to the circulating system would be extremely expensive, the ultimate size of intake structure was initially installed. The intake conduits under the highway from intake structure to the screen house forebay and discharge conduits under highway and lock approach floor were sized for the 2,000 cfs flow. Provision was made to extend the screen house and its forebay for future units. Fig. 8- Flow-Duration Curve shows flow conditions for the period 1934-1954, inclusive.

Individual intake conduits between the pumps and the condenser are monolithic concrete tunnels and are served by 2 - 47,500 gpm vertical pumps. The discharge from 2 condensers is brought together in a single section of the multiple barrel discharge conduit. The location and design of the discharge conduits was coordinated with the Corps of Engineers since it was desirable for them to pass under the lock approach and through the guard wall. Under the lock approach, the multiple barrel discharge conduit



FLOW – DURATION CURVE
PERIOD 1934 – 1954 INCL.

FIGURE 8

separates into 3 - 144' diameter precast concrete pipes. For those locations in the river bed, where it would be possible for a dredge to damage these large pipes, 2' of concrete cover was added. At the end of the pipes, the water is turned in a vertical direction by steel sheet piling cells.

Trash racks and rakes are at the riverside intake while the revolving screens and pumps are across Route 7 on the station property. Water velocities used vary from 4' to 7' per second.

Coal Handling

Coal handling facilities will provide for truck delivery of coal and barge unloading equipment. One of the requirements of the river navigation project was that the river unloading facilities be located back of the locks approach channel, thus placing it a considerable distance upstream from the station. The phase of the coal handling adapted to truck delivery was built first.

The selection of this particular sequence of construction was based on the desire to avoid dredging a channel to the barge unloading equipment. After the New Cumberland Dam is closed, the river pool will be deep enough to allow barge naviagion to the unloading cell without dredging. This scheduling will also avoid the early investment of a large sum of money in facilities which would have a very low use factor until the third and fourth units are completed.

A second conveyor from the breaker house to bunker storage will be installed at the time the navigation coal facilities are built, giving the added reliability of dual facilities.

Local coal stripping operations insure an adequate supply of coal by truck delivery for the first 2 units. Truck coal will be weighed on the site by dual 50-ton truck scales. Tickets will be handled by pneumatic carriers to the drivers so as to speed up truck movement and minimize traffic congestion at the site. Coal from the barge unloader will be transported by a long conveyor consisting of 3 belts in series to a swinging boom stacker, where it will be stacked as active storage over 2 reclaiming hoppers. It can be easily moved from this location to the plant through the breakers. If desired, it can be moved by dozer operation to dormant storage.

Coal storage in bunkers will be 3,000 tons per boiler. Yard storage will be provided for approximately 900,000 tons.

After completion of the coal handling system coal will move from storage through 3 reclaiming hoppers with adjustable feeders to the breaker house where it can be run through either one of the 2 coal breakers. After passing through the breakers, it will be continuously weighed on the belts leading to the bunkers. Cyclic samples will also be taken after it passes through the breakers. All points of discharge are sealed with dust-tight sealing strips. These points are each kept under negative pressure by dust collectors during operation of the conveyor belt system.

Coal at barge unloader and transfer points to swinging boom stacker will also be sprayed with a wetting agent to minimize dust blowing to highway. Principal equipment is rated as follows: Tripper belts - 800 Tph each; barge unloader - 800 Tph.

Chimneys

Chimneys are of reinforced concrete lined with hard-burned shale brick. Each chimney serves 2 boilers and is 503' above grade. Internal diameter at

the top is 21'. The chimneys are designed in accordance with ACI Code 505-54. The inside of the chimney shell is covered with a mastic which in turn receives 2" insulation and then a double brick laid up with a potassium silicate mortar. Fittings such as ladders and rest platforms are of 304 type stainless steel. One set of aircraft warning lights is installed on the circumferential platform near the top of the chimney. These lights are controlled by a photoelectric cell.

Under most operating conditions, the chimneys will be under negative pressure. Usual operating temperatures will be 262 F. Additional thermocouples have been installed at the top of the chimneys to measure the temperature drop in flue gas between entrance to and exit from the chimney. Ambient temperature conditions at the same location will also be measured by thermocouples. Wind velocity and direction instrumentation has also been provided for the purpose of recording the relation of flue gas emission and atmospheric conditions. These instruments are located at top of water tower which is 250' above grade.

Ash Disposal

Fly ash is collected by electrostatic precipitators from which location it is transported by pipe to either the storage silo or to a slurry tank in the pump house. The fly ash silo of about 700 tons capacity was provided to allow the sale of fly ash should a market develop. The ash pump house contains 6 pumps in series for pumping the fly ash over the 500' hill westward of the station to a retention reservoir which will be formed by damming a large ravine on the far side of the hill. By increasing the height of this dam, the ultimate requirements of ash disposal for the station can be met. There are some small areas where ash made during start-up will be dumped to bring the ground elevation above flood stage. Emergency ash disposal ponds are provided to be used in case of need. The disposal of the ash from large generating stations presents an expensive and difficult problem to the designer for solution, especially where bottom land is at a premium as it is along the Ohio River in this location.

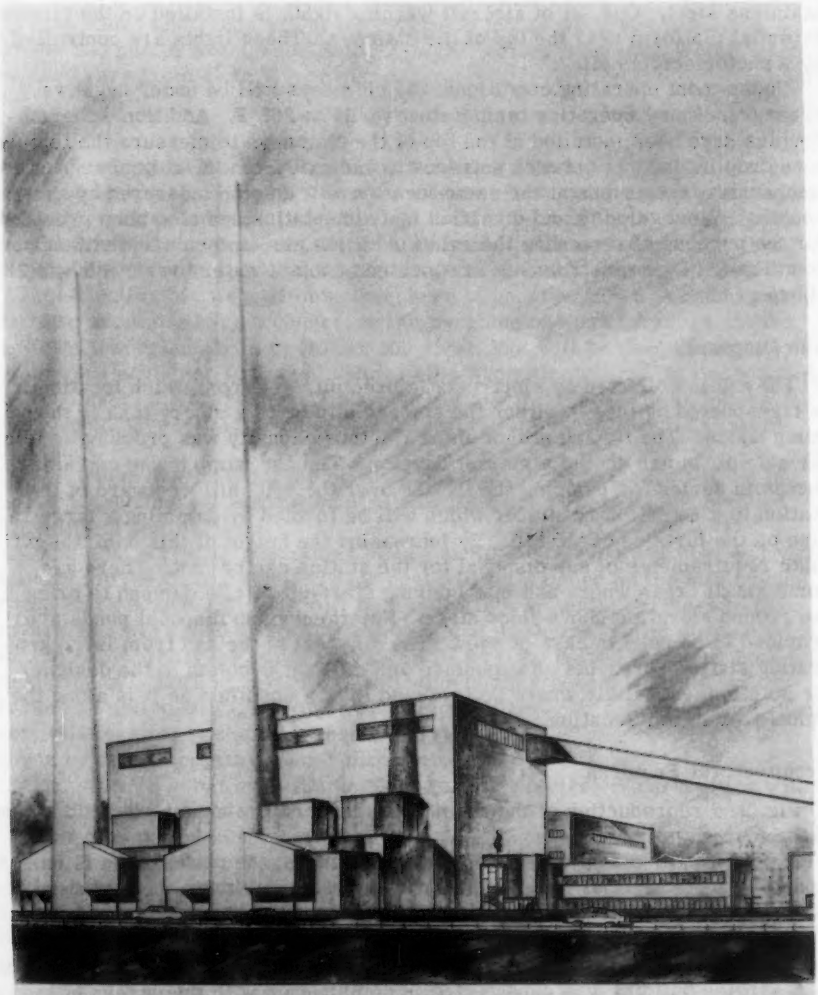
Architectural Features

Fig. 9, a reproduction of the architect's rendering, shows a riverside view of the four unit plant.

Except for the side adjacent to the transformers, the powerhouse is enclosed by a metal panel wall. This wall consists of an inner sheet of steel, 1-1/2" of glass fiber insulation and an outer panel of aluminum.

The office is attached to the powerhouse as a wing and is constructed of masonry.

A service building was constructed in combination with the screen house. It includes heavy and light storage facilities, weightmaster's room, locker room for coal handling crews, paint and oil storage room, hose cart room, chlorine room, tractor repair room and locomotive storage room. A heating boiler was installed in this building which was large enough to provide temporary heat for the powerhouse during construction and start-up of the first 2 units. The service building and the powerhouse are connected with an underground service tunnel through which the various electrical, water, air and heat lines between the 2 buildings are routed.



ARCHITECT'S PERSPECTIVE FIGURE 9

CONCLUSION

The integration of large scale projects into the needs of a growing population and expanding economy requires the skills of many different specialists. The difficult problems of coordination arising must be approached with an open mind and with a spirit of cooperation and a willingness to compromise if these needs are to be fulfilled without unnecessary dissension. Civil engineers along with others on modern project teams must become aware of the need for this cooperative approach as well as their need to keep up with the advancing technology in their own chosen field.

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UNDERGROUND POWER STATIONS IN ITALY

Dante Finzi,[1] Mario Mainardis,[2] and Carlo Semenza[3]

SYNOPSIS

An important role in the development of hydroelectric power in Italy was played by the construction of underground power stations. The reasons for underground plants, statistical data on all of Italy's underground plants and considerations of a general nature are presented in this paper.

According to investigations by the authors, and taking into consideration only power stations the capacity of which is not less than 10,000 kw there are at present 57 underground power stations in operation in Italy.

This figure comprises: 47 underground power stations in the proper sense, called in Italy "in caverna," i.e., an underground excavation connected with the outside through a system of tunnels: we shall call this type simply "underground" (Fig. 1a here below); 3 in pit ("in pozzo") (Fig. 1b); 2 half-underground ("semi-sotterranee")(Fig. 1c); 3 cut away in the rock ("in capanna") (Fig. 1d); 2 buried ("interrate") (Fig. 1e).

Eight more power stations are under construction. Our statistic investigation will therefore comprise 65 power stations.

The construction of the first underground power stations dates from 1925, when work was begun on the Coghinas power station and on the first station of the Upper Flumendosa scheme in Sardinia.

Since those early achievements the construction of this type of works has steadily increased. Construction was particularly intense in the years from 1938 to 1942 and from 1949 to 1953. Until 1935 the over-all capacity installed

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in these types of power stations was about 37,000 kw; then from 1935 to 1940 it rose to 390,000 kw; from 1940 to 1945 to 540,000 kw; from 1945 to 1950 to 1,380,000 kw; and from 1950 to 1955 to 2,996,000 kw.

Including 1957 construction there will be 3,700,000 kw underground, which will correspond approximately to 33% of the whole hydroelectric capacity installed and 26% of the total capacity installed in Italy (i.e., including the thermoelectric and geothermoelectric power station machinery). More than the 30% of the whole Italian hydroelectric capacity is "concentrated" in the 57 stations considered in this paper, this number being only the 2% of the total of 2750 hydro plants in Italy. In other words, the underground power stations have in the average a much higher installed capacity than the outdoor stations.

In 1960 we may expect the capacity installed in the Italian underground power stations to have reached approximately 4,500,000 kw.

Fig. 2 shows the development of the total and underground capacity in Italy from 1927 to 1957. Fig. 3 gives the location of plants by numbers that appear in the list of plants, Table I.

Table I is a list of Italian underground power stations, as of the end of 1957, the capacity of which is greater than 10,000 kw. The table shows their main geometric data and characteristics as well as some data relating to equipment. The list indicates 15 stations, either in operation or under construction, with capacities greater than 100,000 kw and with a total generating capacity of 3,133,900 kw.

Underground power stations, in operation as of June 1957, with installed capacities greater than 50,000 kw number 25 and have a total capacity of 2,840,000 kw.

With regard to the head, the greatest number of underground power stations are middle-head stations (from 140 to 290 m). As for the installed equipment, 20 power stations are equipped with Pelton turbines; 34 with Francis turbines and 6 with Kaplan turbines. S. Massenza power station is equipped with Pelton and Francis turbines; Cotilia power station is equipped with Francis and Kaplan turbines.

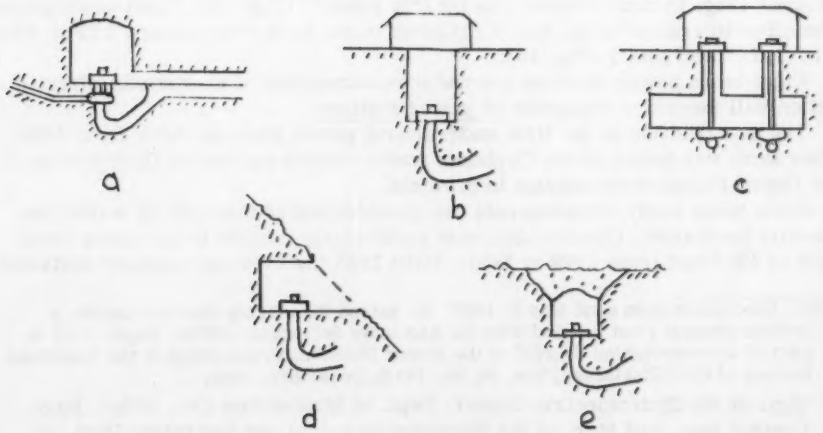


Figure 1. Types of Underground Plants

DEVELOPMENT OF THE TOTAL AND UNDERGROUND CAPACITY IN ITALY FROM 1927 TO 1957

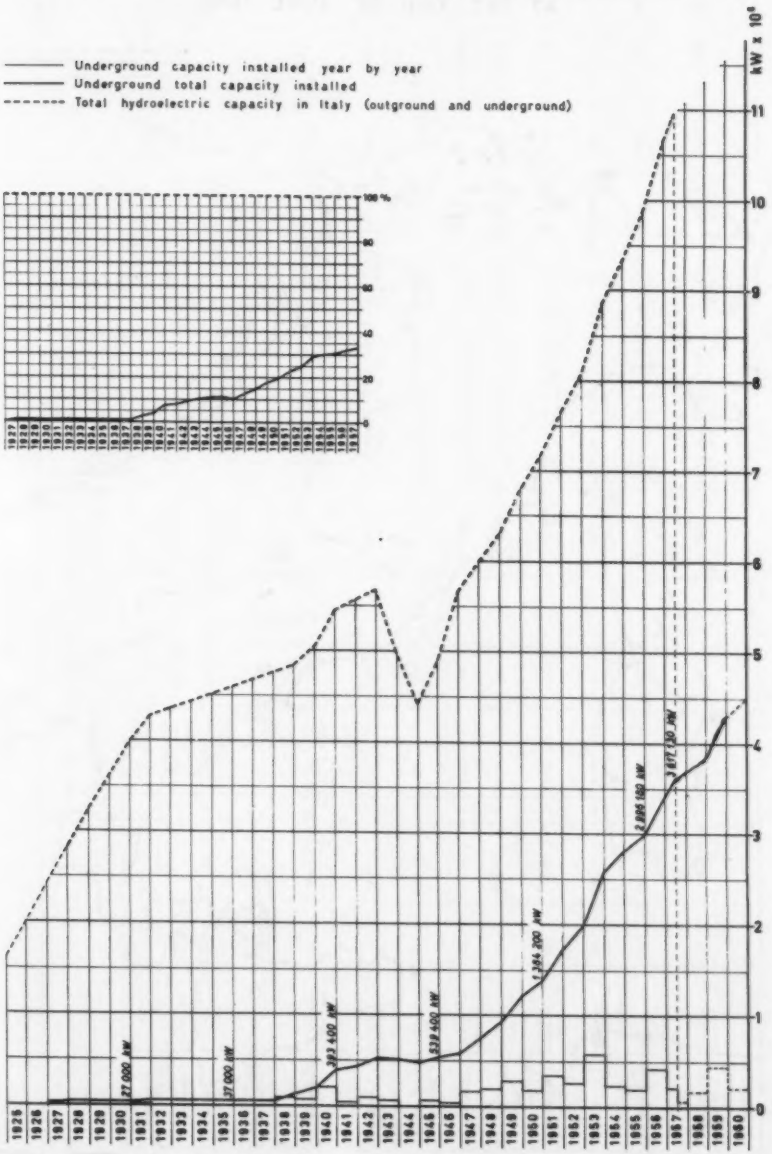

Underground capacity installed year by year
Underground total capacity installed
Total hydroelectric capacity in Italy (outground and underground)
kW x 10⁶
27 000 kW
37 000 kW
393 500 kW
838 500 kW
1 364 200 kW
2 085 100 kW
3 617 130 kW

Fig. 1

UNDERGROUND POWER STATIONS FROM 10000 kW UPWARDS AT THE END OF JUNE 1957

● POWER STATIONS IN OPERATION
○ POWER STATIONS UNDER CONSTRUCTION

Fig. 3

TABLE I

LIST OF THE UNDERGROUND POWER STATIONS IN ITALY
WITH AN INSTALLED CAPACITY FROM 10000 KW UPWARDS

Name	Company To Which the Power Station Belongs	- River Basin - River Harnessed	Max. Flow Rate cm³/sec	Max. Head m	Sets n°	Turbines: Distance Between Centres m	Turbines: Capacity mw	First Operation Year	a) L x W x H b) L_1 x W_1 x H_1	Penstock: No.	Penstock: a) Diameter b) Length m	Penstock: Type	Position of Control Room	Position of Transformers	Entrance Tunnel: Position	Entrance Tunnel: Length	Access Shaft m
1) UNDERGROUND ("IN CAVERNA")																	
a) In Operation																	
1) Coghinas	Elettrica Sarda	-Coghinas -Coghinas	30	98	4	22.00 (1° + 2°s) 15.00 (3° + 4°s)	1) 15 2) 12	1927	a) 82.00 x 15.00 x 22.00 b) 80.00 x 9.00 x 12.00	1	a) 3.00 b) 70	st.,r.c.-c.w.	a.-g.	a.-g.	-	-	70
2) Zapello	Acciaiere e Ferriere Lombarde Falck	-Adda -Livrio, Venina	3.5	400	2	17.795	10	1931	a) 46.55 x 13.00 x 14.45 b) 42.90 x 7.70 x 8.90	1	a) 1.50 ÷ 1 b) 2300	st. - f.t.	Side	a. - g.	Side	142	-
3) Stazzona	Azienda Elettrica Municipale Milano	-Adda -Adda	29	108.29	2	11.00	35	1938	a) 51.50 x 45.30 x 31.00 b) 33.85 x 12.50 x 14.60	1	a) 3.20 b) 130	st. - c.t.	End	i.s.es.	End	110	-
4) Ponte Gardena	Montecantini	-Adige -Isarco	100	61.15	3	13.00	55	1938	a) 68.40 x 19.00 x 28.00 b) 49.50 x 15.00 x 11.50	1	a) 6.00 b) 60	r.c. - c.w.	End	a. - g.	Side	172	-
5) Cencenighe	S.A.D.E.	-Piave -Cordevole, Biois	17.1	202	2	12.00	30	1939	a) 27.60 x 20.40 x 29.20 b) 23.40 x 11.00 x 16.55	1	a) 2.20÷1.83 b) 478	st.- a.g.-c.w.	Side	a. - g.	Side	200	-
6) Doblari	Formerly belonging to S.A.D.E.	-Isonzo -Isonzo	90	45.50	3	11.00	34.5	1939	a) 50.00 x 19.00 x 39.00 b) 37.40 x 13.50 x 15.60	3	a) 3.80 b) 24.00	r.c. - c.t.	End	a. - g.	End	76	-
7) Agordo	S.A.D.E.	-Piave -Cordevole, Corpassa	25	150	2	9.00	26	1940	a) 34.70 x 15.20 x 28.35 b) 30.80 x 12.00 x 16.05	1	a) 2.40 ÷ 2.20 b) 200	st. - f.t.	End	a. - g.	Side	25	-
8) Bressanone	Ferrovie dello Stato	-Adige -Isarco, Rienza, Rio Valles	72	164	5	12.45	142.5	1940	a) 108.30 x 20.50 x 32.60 b) 82.92 x 15.00 x 16.70	1	a) 5.80÷4.70 b) 350	r.c. - c.t.	End	a. - g.	End	60	-
9) Piava	Formerly belonging to S.A.D.E.	-Isonzo -Isonzo	80	26.50	2	13.00	17.4	1940	a) 50.20 x 19.00 x 42.60 b) 30.00 x 12.80 x 17.95	2	a) 5.20÷3.60 b) 62.35	r.c. - c.t.	End	i.m.e.	End	126	-

TABLE I

Plate 16, Sheet 2 of 6

Name	Company To Which the Power Station Belongs	- River Basin - River Harnessed	Max. Flow Rate	Max. Head	Sets	Turbines			First Operation Year	a) L x W x H b) L_1 x W_1 x H_1	Penstock			Position of Control Room	Position of Transformers	Entrance Tunnel		Access Shaft
						Distance Between Centres	Capacity				No.	a) Diameter b) Length	Type			Position	Length	
			m	m	n°	m	mw					m					m	m
10) Salisano	Azienda Comunale Elettricita Acque Roma	-Tevere -Peschiera, Velino	8	244.55	2	7.00	16		1940	a) 48.00 x 15.00 x 18.75 b) 39.00 x 9.00 x 10.70	2	a) 1.40+1.10 b) 490	st. - f.t.	i.m.e.	a. - g.	Side	130	-
11) Regoledo 4° salto Bitto	Orobia	-Adda -Bitto di Gerola, Alto Varrone	8.40	482.65	2	a) 6.80 b) 14	36		1941	a) 61.00 x 12.00 x 35.00 b) 44.90 x 10.00 x 10.00	1	a) 1.60+1.40 b) 889.60	st.-a.g.,f.w. f.t.	End	i.s.es.	End	-	-
12) Comunacqua	Consorzio Idro-Elettrico Alto Aniene e Simbrivio	-Tevere -Aniene, Simbrivio	8	135.20	2	7.00	10		1942	a) 22.50 x 10.80 x 21.50 b) 21.30 x 9.00 x 10.50	1	a) 2.00 b) 200	r.c. - c.t.	a. - g.	a. - g.	End	120	-
13) Grigno	Industriale Trentina	-Brenta -Brigno	6.2	507.40	2	9.85	12		1942	a) 52.50 x 12.10 x 14.30 b) 38.80 x 7.50 x 8.80	1	a) 1.00+0.95 b) 674	st. - f.t.	End	i.s.es.	End	28	-
14) Stanga	S.A.D.E.	-Piave -Cordevole, Sarsana	23	152	2	10.00	30		1943	a) 39.20 x 19.00 x 30.85 b) 27.00 x 11.00 x 15.90	1	a) 2.85 b) 152.40	r.c.,st.-c.w., c.t.	End	a. - g.	Side	55	-
15) S.Giacomo 2° salto Vomano	Terni	-Vomano -Vomano	37.5	663	3	a) 11.80 b) 24.50	165		(1°s.) 1947 (2°s.) 1948 (3°s.) 1950	a)100.10 x 38.80 x 23.90 b) 96.60 x 16.30 x 14.50	1	a) 2.50 b) 590	st. - c.w.	Side	a. - g.	Side	1820	654
16) Belviso Inferiore	Acciaierie e Ferriere Lombarde Falck	-Adda -Belviso, Aprica, Caronella, Bondone	14.5	547.30	2	a) 9.70 b) 20.00	70		(1°s.) 1947 (2°s.) 1949	a) 92.75 x 30.00 x 25.50 b) 56.00 x 14.00 x 13.60	1	a) 2.00+1.65 b) 924	st. - a.g., f.t.	End	i.m.e.	End	36.75	-
17) Ampezzo	S.A.D.E.	-Tagliamento -Lumiei, Novarza, Tagliamento	16.50	480.65	3	7.36	60		1948	a) 72.00 x 19.50 x 25.30 b) 52.00 x 12.00 x 13.25	1	a) 1.80+1.69 b) 719.065	st. - a.g., f.w., f.t.	End	a. - g.	End	118	-
18) Glorenza	Montecatini	-Adige -Resia, Puni	18	589.35	2	a) 11.95 b) 19.25	100		(1°s.) 1948 (2°s.) 1949	a) 57.00 x 19.70 x 24.50 b) 54.60 x 13.70 x 11.20	1	a) 2.50+2.00 b) 992	st. - f.t.	a. - g.	a. - g.	End	500	-
19) Castelbello	Montecatini	-Adige -Adige, Plima, Lasa, Ramini	30	294.70	3	9.50	85		1949	a) 60.00 x 18.00 x 33.00 b) 40.55 x 13.50 x 18.10	1	a) 3.25+2.80 b) 566	r.c. - c.t.	End	a. - g.	End	190	-
20) Flumendosa 2° salto	Elettrica Sarda	-Alto Flumendosa -Flumendosa	7	418	3	a) 6.34 b) 12.80	29.56		1949	a) 63.00 x 15.00 x 15.40 b) 49.00 x 13.00 x 13.70	1	a) 1.90+1.20 b) 1200	st. - a.g., c.w.	a. - g.	a. - g.	End	-	-
21) Flumendosa 3° salto	Elettrica Sarda	-Flumendosa -Sa Teula	7	171	2	9.50	10.04		1949	a) 29.60 x 9.00 x 12.10 b) 25.50 x 7.50 x 10.80	1	a) 1.40 b) 357	st. - a.g., c.w.	a. - g.	a. - g.	-	-	41

21) Flumendosa 3° salto	Elettrica Sarda	- Flumendosa -Sa Teula	7	171	2	9.50	10.04			b) 25.50 x 7.50 x 10.80		b) 397	c.c.						

TABLE I

Plate 16, Sheet 6 of 6

Name	Company To Which the Power Station Belongs	- River Basin - River Harnessed	Max. Flow Rate	Max. Head	Sets	Turbines			First Operation Year	a) L x W x H b) L_1 x W_1 x H_1	Penstock			Position of Control Room	Position of Transformers	Entrance Tunnel		Access Shaft
						Distance Between Centres	Capacity				No.	a) Diameter b) Length	Type			Position	Length	
			m	m	n°	m	mw					m					m	m
22) Provvidenza 1° salto Vomano	Terni	-Vomano -Vomano, Tronto	60	267.50	3	31.43	150		(1°s.) 1949 (2°s.) 1953	a) 128.70 x 18.80 x 25.00 b) 119.70 x 16.00 x 15.50	1	a) 4 ÷ 2.50 b) 265	st. - c.w.	End	i.m.s.	End	629	-
23) Chiavenna 1° salto Mera	Edison	-Adda -Mera	20	335.30	3	8.00	60.45		(1°s.) 1949 (2°s-3°s.) 1950	a) 35.00 x 14.00 x 27.00 b) 31.50 x 10.60 x 14.00	1	a) 2.55 ÷ 2.25 b) 506.70	st. - c.t.	a. - g.	a. - g.	End	42	-
24) Pontey	Idroelettrica Piemonte	-Dora Baltea -Dora Baltea	55	69.50	2	12.00	33		(1°s.) 1950 (2°s.) 1951	a) 53.70 x 20.50 x 37.00 b) 36.20 x 12.50 x 19.50	1	a) 3.80 b) 80	r.c. - c.t.	End	i.s.e.	2 Sides	40	-
25) Cedegolo	Edison	-Oglio -Rabbia, Remula, Poglia	24	234	3	11.00	73.2		(1°s.) 1950 (2°-3°s.) 1951	a) 47.00 x 16.90 x 28.00 b) 43.50 x 13.80 x 13.60	1	a) 3.50 ÷ 2.60 b) 319.80	st. - a.g., c.w.	a. - g.	a. - g.	End	337	-
26) Monte Argento	Terni	-Tevere -Nera	150	51.60	3	16.00	66		(1°s.) 1950 (2°-3°s.) 1951	a) 80.00 x 13.00 x 34.00 b) 64.00 x 10.40 x 15.00	3	a) 3.80 b) 65	r.c. - c.w.	End	a. - g.	-	-	49.60
27) S. Giustina	Edison	-Adige -Noce	66	184.50	3	12.50	103.50		1951	a) 68.50 x 21.30 x 34.00 b) 56.85 x 13.70 x 16.90	1	a) 3.70 ÷ 3.50 b) 151	st. - f.t.	a. - g.	a. - g.	End	325	-
28) Soverzene	S.A.D.E.	-Piave -Piave	88	284	4	14.00	220		(1°-2°s.) 1951 (3°s.) 1952 (4°s.) 1956	a) 74.00 x 25.40 x 39.20 b) 68.60 x 14.65 x 18.35	4	a) 2.55 b) 225	p.r.c. - 2 c.t.	Side	i.s.e.	Side	535	-
29) Signayes	Consorzio Elettrico del Buthier	-Dora Baltea -Buthier	16	347	3	a) 6.50 b) 12	42.6		(1°s.) 1951 (2°-3°s.) 1953	a) 50.00 x 17.00 x 17.00 b) 42.00 x 13.00 x 12.00	1	a) 1.65 b) 700	st. - f.t.	a. - g.	a. - g.	-	-	118
30) Villa S. Maria	Consorzio Idroelettrico del Sangro	-Sangro -Sangro	16	424.79	3	8.32	72.9		(1°-2°s.) 1952 (3°s.) 1953	a) 98.00 x 19.50 x 22.40 b) 78.55 x 13.50 x 12.30	1	a) 2.10 b) 410	st. - f.w.	a. - g.	i.s.e.	End	385	-
31) S. Massenza	Idroelettrica Sarca e Molveno	-Mincio -Sarca, Bondai	1) 70 2) 15	1) 590 2) 202.70	1) 6 2) 1	18.70 ÷ 25.81	375		1952 1953 1954 1957	a) 92.70 x 28.80 x 28.00 b) 88.20 x 15.30 x 15.80	1) 2 2) 1	1a) 2.68 ÷ 2.55 b) 733 ÷ 834 2a) 2.40 b) 300	st. - c.t.	Side	i.m.s.	End	360	-
32) Acri 1° salto Mucone	Meridionale di Elettricita	-Crati -Mucone	20	637.55	2	10.05	110		1953	a) 78.00 x 26.50 x 25.00 b) 72.00 x 15.70 x 13.35	1	a) 2.50 ÷ 2.00 b) 843.46	st. - f.t.	End	i.s.e.	End	800	-
33) Isolato 1° salto Liro	Edison	-Adda -Liro	7.75	654.45	2	a) 7.76 b) 15.56	42.8		1953	a) 42.00 x 13.00 x 21.50 b) 40.40 x 11.35 x 11.00	1	a) 1.85 ÷ 1.40 b) 1543.47	st. - a.g., c.t.	a. - g.	a. - g.	End	220	-
34) Morasco	Edison	-Ticino -Sabbione, Nefelgiu	8.14	640.35	2	15.00	44.7		1953	a) 54.40 x 16.50 x 20.70 b) 39.40 x 14.50 x 10.00	1	a) 1.60 ÷ 1.35 b) 984.65	st. - a.g., f.t.	End	i.s.e.	End	175	-

TABLE I

Plate 16, Sheet 9 of 6

Name	Company To Which the Power Station Belongs	- River Basin - River Harnessed	Max. Flow Rate m	Max. Head m	Sets n°	Turbines: Distance Between Centres m	Turbines: Capacity mw		First Operation Year	a) L x W x H b) L_1 x W_1 x H_1	Penstock: No.	Penstock: a) Diameter b) Length m	Penstock: Type	Position of Control Room	Position of Transformers	Entrance Tunnel: Position	Entrance Tunnel: Length m	Access Shaft m
35) Ala	Elettrica Ala	-Adige -Adige	200	27.95	2	17.00	45		1953	a) 44.00 x 25.70 x 40.30 b) 42.00 x 13.50 x 16.60	2	a) 5.60 b) 9.00	r.c. - c.t.	Side	a. - g.	End	96	-
36) Lana	Trentina di Elettricita	-Adige -Valsura, Marano	30	485.60	3	a) 8.30 b) 16	120		(1°-2°s.) 1953	a) 71.60 x 17.00 x 25.10 b) 66.90 x 13.20 x 17.50	1	a) 2.60+2.40 b) 1256.30	st.-a.g.,c.t.	a. - g.	a. - g.	End	270	-
37) Avise	Idroelettrica Piemonte	-Dora Baltea -Valgrisanche	16.7	1036	3	17.50(1°+2°s.) 21.35(2°+3°s.)	146.7		(1°-2°s.) 1954 (3°s.) 1957	a) 79.10 x 25.90 x 26.60 b) 70.00 x 15.00 x 14.65	1	a) 1.90+1.50 b) 1529	st. - f.t.	a. - g.	i.m.e.	End	660	-
38) Cavilla	Selt - Valdarno	-Brenta -Cismon	40	82	2	10.00	35		1954	a) 34.00 x 17.50 x 32.00 b) 32.50 x 12.50 x 13.00	1	a) 3.40 b) 36.00	r.c. - c.t.	a. - g.	a. - g.	Side	260	-
39) Pantano D'Avio	Edison	-Oglio -Avio	3.8	446.5	2		14.1		1955		1	a) 1.25+1.00 b) 617						-
40) Arsie	Selt - Valdarno	-Brenta -Cismon, Senaiga	30	141	2	10.00	48		1955	a) 33.00 x 17.00 x 34.00 b) 30.00 x 12.50 x 13.00	1	a) 3.25 b) 91.50	r.c. - c.t.	a. - g.	a. - g.	Side	170	-
41) Cardona	S.A.D.E.	-Piave -Mae	14.5	150	1	-	15		1955	a) 35.10 x 14.55 x 20.40 b) 32.95 x 11.40 x 11.05	1	a) 2.00 b) 95	p.r.c. - st. - c.w., c.t.	End	a. - g.	Side	120	-
42) Montorio al Vomano - 3° salto al Vomano	Terni	-Vomano -Vomano, Castellano	52	262.40	3	11.00	116.2		1955	a) 63.00 x 22.50 x 32.50 b) 60.00 x 14.50 x 19.30	1	a) 3.10 b) 283	st. - c.w.	i.s.e.	i.s.e.	End	900	-
43) Malga Ciapela	S.A.D.E.	-Piave -Adige, Ombretta, Avisio	4	567	1	6.80	20		1956	a) 47.90 x 14.40 x 21.50 b) 32.00 x 10.00 x 12.30	1	a) 1.30+1.05 b) 723	st. - f.t.	End	i.m.e.	End	156	-
44) Premadio	Azienda Elettrica Municipale - Milano	-Adda -Adda, Frodolfo	65	708	5	a) 10.80 b) 22.50	375		(1°-2°s.) 1956	a) 56.20 x 33.20 x 30.50 b) 47.20 x 16.30 x 17.95	1	a) 2.60+2.30 b) 853	st. - f.t.	a. - g.	i.m.e.	Side	350	-
45) S. Floriano D'Egna	Avisio	-Adige -Avisio	30	571	3	a) 9.70 b) 17.00	195		(1°-2°s.) 1956 (3°s.) 1957	a) 78.10 x 27.70 x 25.60 b) 74.90 x 12.00 x 12.90	1	a) 2.60+2.40 b) 750	st. - f.t.	Side	a. - g.	Side	240	-
46) Nembia	Idroelettrica Sarca e Molveno	-Sarca -Sarca	27	88 + 22	1	-	13.5		1957	a) ø = 18.60;h = 62.80 b) ø = 16.96;h = 41.20	1	a) 3 + 2.30 b) 140	r.c.,st. - c.t.	a. - g.	a. - g.	End		30.25
47) Somplago	S.A.D.E.	-Tagliamento -Tagliamento, Lumiei, Ambiesta	110	290	5	14.00	275		(1°-2°s.) 1957	a) 90.40 x 27.80 x 36.70 b) 85.20 x 16.80 x 17.35	5	a) 2.55 b) 221	st. - 2 f.w.	a. - g.	i.s.es.	Side	512	-

TABLE I

Plate 16, Sheet 5 of 6

Name	Company To Which the Power Station Belongs	- River Basin - River Harnessed	Max. Flow Rate m	Max. Head m	Sets n°	Turbines: Distance Between Centres m	Turbines: Capacity mw		First Operation Year	a) L x W x H b) L_1 x W_1 X H_1	Penstock: No.	Penstock: a) Diameter b) Length m	Penstock: Type	Position of Control Room	Position of Transformers	Entrance Tunnel: Position	Entrance Tunnel: Length m	Access Shaft m
b) Under Construction																		
48) Brunico	Idroelettrica Atesina	-Adige -Rienza	22	204.30	2	11.20	21		1958	a) 38.70 x 19.10 x 31.70 b) 29.20 x 12.60 x 12.80	1	a) 2.70 b) 260	p.r.c. - c. t.	End	i.s.es.	End	360	-
49) Colomber	S.A.D.E.	-Piave-Piave -Vaiont-Mae	20	133.50	1	-	10		1959	a) 32.30 x 18.60 x 22.30 b) 26.00 x 13.00 x 13.20	1	a) 3.00 b) 30	st. - c.t.	End	a. - g.	Side	28	-
50) Grosio	Azienda Elettrica Municipale - Milano	-Adda -Adda, Viola, Frodolfo	80	598.20	4	24.50	400		1959	a) 22.00 x 20.60 x 33.90 b) 14.20 x 15.60 x 15.00	2	a) 3.30+2.70 b) 985	st. - f.t.	a. - g.	i.s.e.	Side	627	-
51) Malga Boazzo 1° salto Alto Chiese	Idroelettrica Alto Chiese	-Oglio -Alto Chiese		568	2		96											
52) Narni	Terni	-Nera -Nera	180	28	2	18.00	44.6		1958	a) 58.70 x 21.00 x 34.80 b) 56.70 x 14.00 x 13.50	2	a) 5.35 b) 40	r.c. - c.t.	a. - g.	a. - g.	Side	140	-
53) Quart	Idroelettrica Piemonte	-Dora Baltea -Dora Baltea	60	75.20	1	11.80	40		1958	a) 47.00 x 23.00 x 23.70 b) 42.50 x 15.00 x 17.25	1	a) 4.00 b) 94	st. - c.t.	a. - g.	a. - g.	Side	98	-
54) S. Silvestro	Selt - Valdarno	-Brenta -Noana	14	450	1	-	60		1958	a) 30.00 x 13.80 x 27.00 b) 19.40 x 11.00 x 14.68	1	a) 1.30 b) 484.57	st. - c.t.	i.m.e.	a. - g.	End	53.30	-
55) Sondrio 2° salto Mallero	Vizzola	-Adda -Mallero	26.5	663.70	2	14.00	140		1959	a) 43.70 x 28.00 x 29.50 b) 41.90 x 17.80 x 14.10	1	a) 2.60+2.20 b) 1400	st.-a.g.,c.t.	a. - g.	a. - g.	End	115	-
11) CUTAWAY																		
56) Hone	Idroelettrica Piemonte	-Dora Baltea -Dora Baltea	55	36.87	1	-	18.5		1947	a) 28.00 x 21.60 x 39.75 b) 22.85 x 13.00 x 17.25	1	a) 4.50 b) 66	r.c. - c.t.	Side	i.s.e.	End	-	-
57) Lovero	Azienda Elettrica Municipale - Milano	-Adda -Adda, Roasco	30	110.15	2	12.00	44		1948	a) 53.65 x 41.50 x 31.00 b) 38.00 x 12.50 x 14.80	1	a) 3.50 b) 207.40	r.c. - c.t.	Side	i.m.e.	Side	-	-
58) Barcis	S.A.D.E.	-Livenza -Cellina	45	61.40	2	11.00	20		(1°s.) 1954	a) 27.00 x 21.00 x 36.00 b) 21.30 x 13.00 x 10.65	2	a) 5.00; 2.50 b) 19	r.c. st. - c.w c.t.	i.m.e.	i.m.e.	Side	-	-

TABLE I

Plate 16, Sheet 5 of 6

Name	Company To Which the Power Station Belongs	River Basin - River Harnessed	Max. Flow Rate m³/s	Max. Head m	Units n°	Turbines: Distance Between Centres m	Turbines: Capacity mw	First Operation Year	a) L x W x H b) L_1 x W_1 x H_1	Penstock: No.	Penstock: a) Diameter b) Length m	Penstock: Type	Position of Control Room	Position of Transformers	Access Entrance Tunnel: Position	Access Entrance Tunnel: Length m	Access Shaft m
III) PIT																	
59) Cividate	Edison	-Oglio -Oglio	35	130.45	3	11.00	52.5	1945	a) 62.50 x 20.00 x 44.00 b) 53.30 x 6.00 x 7.30	1	a) 3.20 b) 250.60	st. - a.-g. f.t.	a.-g.	a.-g.	-	-	-
60) Meduno	Agricola Industriale Cellulosa Italiana	-Livenza -Meduna	20	71.27	3	5.30	13.68	(1°-2°s.) 1952	a) ϕ = 18.50; h = 57.25 b) ϕ = 16.50; h = 44.00	1	a) 3.00 b) 74	r.c. - c.t.	a.-g.	a.-g.	-	-	44
61) Vinadio	Compagnia Imprese Elettriche Liguri	-Tanaro -Stura di Demonte	18	385	2		61.25	1956		1	a) 2.70+2.08 b) 631.59	st.-f.t.-a-g.	a.-g.	a.-g.			
IV) HALF UNDERGROUND																	
62) Triano 4° salto Pescara	Meridionale di Elettricità	-Aterno, Pescara -Pescara	60	48.35	3	12.50	26.25	1942	a) 76.25 x 27.20 x 38.00 b) 37.50 x 9.40 x 5.20	3	a) 2.90 b) 200	r.c. - c.t.	a.-g.	a.-g.	-	-	26.15
63) Montelungo	Meridionale di Elettricità	-Volturno -Volturno	30	140.30	2	13.20	32.5	(1°s.) 1953 (2°s.) 1954	a) 28.50 x 18.00 x 26.45 b) 39.38 x 11.70 x 13.00	2	a) 2.70 b) 350	st. - f.t.	a.-g.	a.-g.	-	-	19.25
V) BURIED																	
64) Cotilia	Terni	-Tevere 1) Turano, Salto 2) Alto Velino 3) Canetra 4) Peschiera	1)-2) 50 3) 11 4) 33.4	1)-2) 145.59 3) 32.6 4) 8.64	4	1)-2) 20.82	1) 2) 60 2) 3.2 3) 3.1	1) 1942 2) 1946 3) 1952 4) 1943	a) 14.00 x 36.00 x 28.00 b) 93.00 x 17.40 x 18.00	4	1)2a) 4x2.50 b) 248 2a) 2.00 b) 700 4a) 4.70 b) 15	1)3)3)r.c.-st. - c.t. 4) r.c. - a-g.	End	a.-g.	End	26	-
65) Gorzia Alta 2° salto Bitto	Orobia	-Adda -Bitto di Gorzia	2.48	715.40	1	-	13.5	1942	a) 31.75 x 15.30 x 20.00 b) 19.50 x 10.50 x 9.00	1	a) 1.06+0.77 b) 1899.15	st. - f.t.-a-g.	End	a.-g.	Side	10.00	-

Nomenclature:

st.	= steel	End	= at one end of the room	P= Pelton	
r.c.	= reinforced concrete	Side	= along the sides	F= Francis	
p.r.c.	= prestressed reinf. conc.	a.-g.	= aboveground	K= Kaplan	
f. t.	= free in tunnel	i.m.e.	= inside main excavation	V= vertical	
c.t.	= concreted in tunnel	i.s.e.	= in separate excavation	O= overhung	
f. w.	= free in well	i.s.e.	= in separate excavations	d= double overhung	
c. w.	= concreted in well	s.	= set		

The map, Fig. 3, shows that 50 power stations are situated in North Italy, 5 in Central Italy, 7 in South Italy and 3 in Sardinia.

Reasons for the Adoption of Underground Power Stations

In Italy, the adoption of underground power stations is considered as being due to several reasons; they have also followed the evolution of construction principles and the variations in the costs of material.

The first underground power stations, at Coghinas (put into operation in 1927), Pian Sule (1929-1931), Foce Ponale (1934), Vizza (1938-1940) etc., were placed underground mostly because their locations were in areas hardly accessible on account of steep rocky walls or unsuitable for objective conditions such as the danger of avalanches. The position of these power stations was therefore dictated by principles of a topographic nature, and not of more general character as are the principles that govern the construction of these power stations at present.

In the period preceding the second world war, defense reasons came also to the foreground, especially for the plants built near the frontier.

Since that period, however, the choice between the underground solution and the outdoor solution was mainly determined by economic reasons; namely by the economic and technical evaluation of all the factors—both individually and as a whole—that intervene in such comparisons. These factors may be divided into three groups: (a) the general layout of the scheme; (b) the morphological and general characteristics of the superficial ground; and (c) construction and hydrodynamic reasons. Defense reasons (d) are still important, although the other factors are determined in establishing an economic comparison between the two solutions. We shall now examine briefly each of the foregoing points.

(a) The point which is now generally regarded as a basis for choosing or discarding the underground solution is the general layout of the scheme. The problem of the power station cannot be separated from that of the whole scheme, with which it has to be studied and solved. Now, from this point of view, the underground solution allows, to a certain extent, to place the power station where it is most convenient for the general layout. In other words, if the superficial features of the ground are not very favorable to the building of the power station and connected structures (such as surge tanks, penstocks and outlet canals) and if the characteristics of the rock are satisfactory, we are able—by adopting the underground solution—to dispose of a much wider field in which to place the power station. Thus it will be easier for us to interrupt a head at the most suitable point or extend the supply works until the most convenient degree of exploitation is reached, or also somewhere collect in one power station supply works proceeding from different catchment areas with different heads.

The above principle has already become, for most designers, the fundamental factor in designing and building power schemes, i.e., the opportunity or necessity of overlooking local conditions and conceiving the schemes spatially. This principle, however, must not be taken in an absolute sense, but rather in a relative one for, of course, the features both of the superficial ground and the rock come into play.

(b) With regard to the outground morphological conditions we can only say that the reasons which determined the construction of the first underground

power stations are still in force. In fact, whenever the access to a given location is difficult or the area is exposed to the danger of avalanches or other risks, or again the power station cannot, or can hardly be placed outdoors due to the insufficiency of the ground (which could be more profitably used for housing, etc.), the construction of underground power stations appears to be the ideal solution to the problem.

(c) Besides the basic elements previously mentioned, the economic comparison of the outdoor solution with the underground solutions is determined by numerous other factors of a construction and hydrodynamic nature, and should therefore be carried out case by case.

First of all, as far as the power station proper is concerned, it must be assumed that the underground solution is often by itself more expensive than the outdoor one for various reasons:

(i) Excavation, though it does not constitute a high percentage of costs, is usually by itself an addition to the other works indispensable for the construction of the power station.

(ii) The volumes of masonry can be practically much the same in the two solutions; yet, in the underground solution, the lining is often double—i.e., against the rock side and on the inside—and the arched lining of roof is more expensive than any external covering. On the other hand, the saving in machinery foundations can be considerable, if the outdoor solution entails foundations on "loose" ground. In this connection it should be underlined that the tendencies to leave rock blocks within the masonry for economy's sake has to be regarded with caution. Even if the rock is solid, the discontinuity in the planes of foundation may cause small settlements and fissures clearly unfavorable to the foundations of the turbine-generator sets. If the foundation in the outdoor solution is also on the rock, the cost of the underground solution is obviously higher.
Nevertheless the underground solution is more advantageous in so far as work can be carried on independently of seasonal conditions: more reliable construction schedules can therefore be planned.

(iii) The various constructive elements and accessory services in the underground power station are expensive, while in the outdoor solution they do not exist or have a limited importance (access tunnel, outlet tunnel, tunnels for bus bars, cables, air conditioning, transportation, etc.). These are the elements that can really affect cost more than the main excavation itself.

So far we have mostly mentioned the economic drawbacks of the underground solutions. Yet front to front with these are first the advantages related to the general plan of the scheme as mentioned at point a), and second some economic advantages relating to adjoining structures (penstocks, surge tanks, equipment, etc.) which must be evaluated case by case and can be major factors in the economics of adopting the underground solution.

On this last subject, first of all we should consider the possibility of reducing the length of the penstocks to a minimum due to the abolition of the relief valves, and the dimensions of the surge tanks due to the shorter length of pressure conduits. In modern plants where the utilization of sensible level oscillations in reservoirs is of great importance, the reduction of the penstock length to a minimum makes it possible to face operations of the units that would otherwise be very difficult. In many big powerhouses it was possible,

through the adoption of the underground solution, to satisfy without any excess of expenditure very severe conditions for the operation of the units and to eliminate, as pointed out above, the relief valves. For instance at Soverzene, relief valves would have been very expensive on turbines with a capacity of 22 m^3/sec on a head varying from 285 to 225 m. Thus a considerable saving was achieved, besides the one resulting from the reduction of the penstock length.

Moreover, through the adoption of the underground solution it is possible to achieve considerable savings by utilizing fully the resistance of the rock for penstocks. On this subject we note that in Italy, like in all countries where the iron production is rather poor, the incidence of penstock costs on the total cost of a plant is remarkable; efforts are therefore made to reduce such costs, which has in some cases led to the discard of solutions with all steel penstocks (Soverzene). In minimizing the tonnage of steel, the possibility of reducing the length of penstocks to a minimum is also important.

The same advantage can be gained in the surge tanks which can, by this solution, be placed at a considerable depth into the rock according to the position of the underground power station, thus often obtaining, from the point of view of rock resistance, more favorable and cheap conditions than those that would occur, should the surge tanks be placed near the surface.

No absolute rule can actually be established in all these connections, but only an objective balance of technical and economic reasons.

(c) The defense factor is still an important one: we are well aware of the fact that Italian recovery after the war was considerably slowed down by the destruction of numerous hydroelectric plants, carried out by the retreating armies in the central regions in 1945 and after. The hydraulic structures of the plants had generally undergone little damage, but most of the equipment of the power stations had been destroyed. The capacity in Central Italy has been reduced to 15% of prewar availability.

Although the greatest damage was caused by pioneers rather than by bombing or shelling, this can by no means be regarded as a general rule, denying the importance of the underground location as a protection factor in power stations.

From the point of view of safety, it may be remembered that 15 years ago an 8 to 10 m thickness of good rock was considered by experts to be a

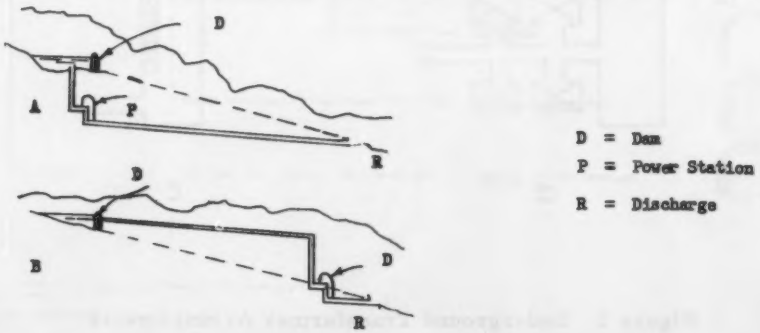


Figure 4. Types of Underground Schemes

sufficient protection against any ordinary war offense. We lack sufficient elements by which to judge the effects of atomic bombs, but we believe that a significantly thicker layer of compact rock can be a substantial factor in this respect.

Different Types of Underground Hydroelectric Schemes

After setting forth the reasons why the construction of underground power stations is frequently adopted in modern technique, let us examine briefly the various types of possible underground plants.

Underground hydroelectric schemes may be reduced to the two following types, or a combination of them (Fig. 4):

Case A

(1) The sides of the valley are not sufficiently high to allow the drilling of a supply tunnel at the dam foot elevation;
(2) The rock in which the outlet canal is drilled being waterproof, no considerable infiltration of water from the river is expected during excavation;
(3) The outlet canal can be divided into various portions by means of adits of reasonable depth.

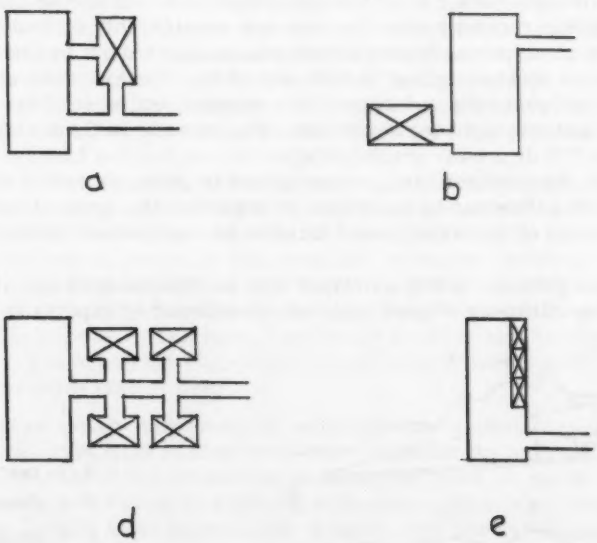
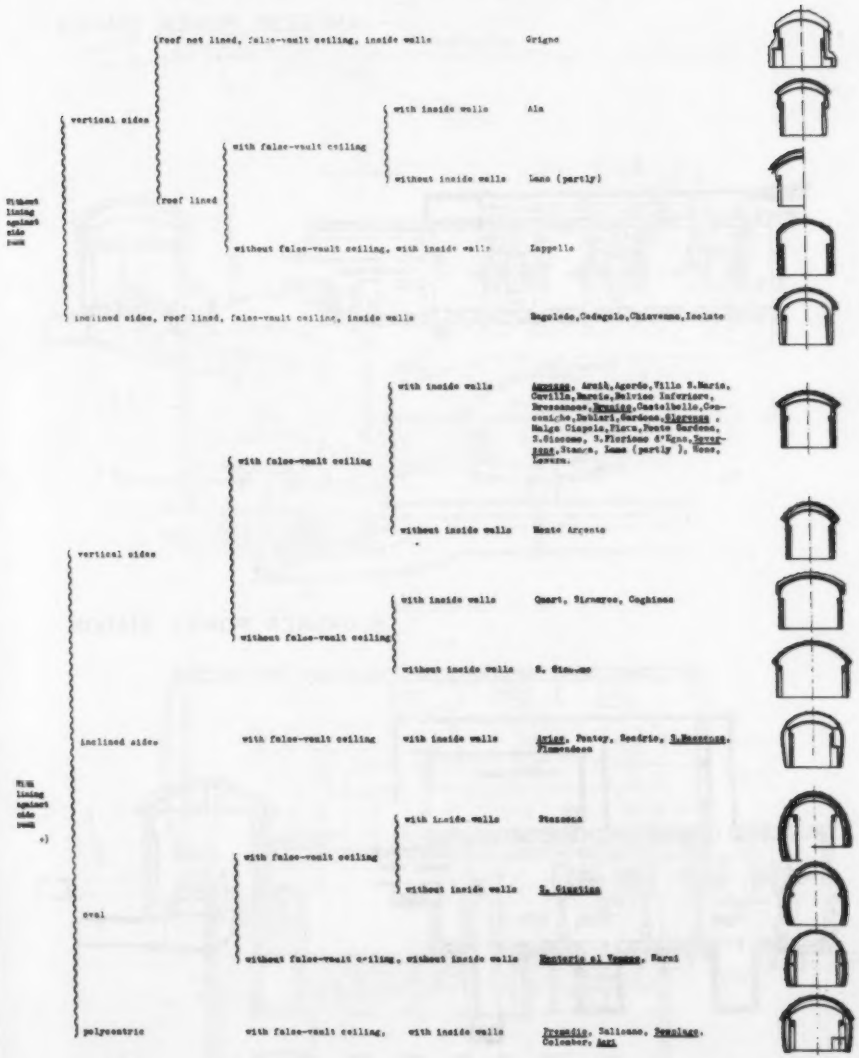


Figure 5. Underground Transformer Arrangements

Cross section shapes and lining features of the main underground power stations in Italy



Note: A plate on the underlined power stations is enclosed.

*) (either of unreinforced or reinforced concrete)

Fig. 6

AMPEZZO POWER STATION

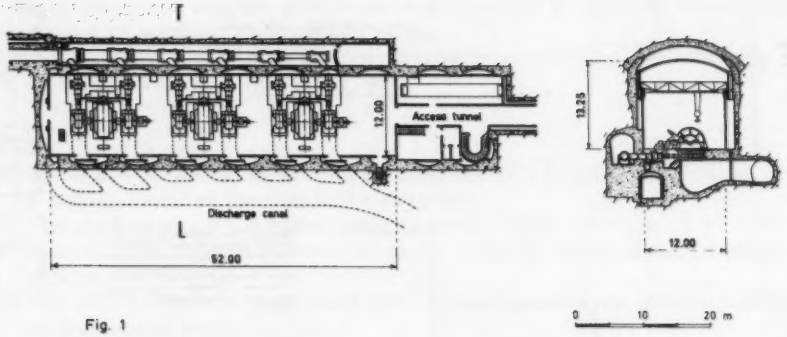


Fig. 1

GLORENZA POWER STATION

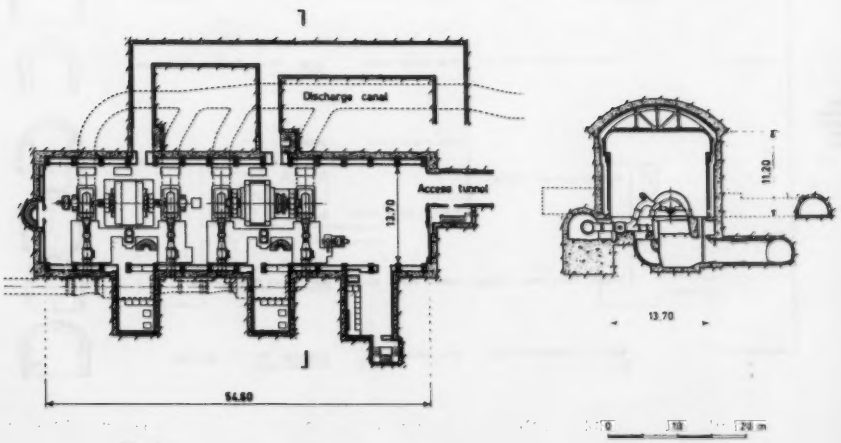


Fig. 2

Fig. 7

SOVERZENE POWER STATION

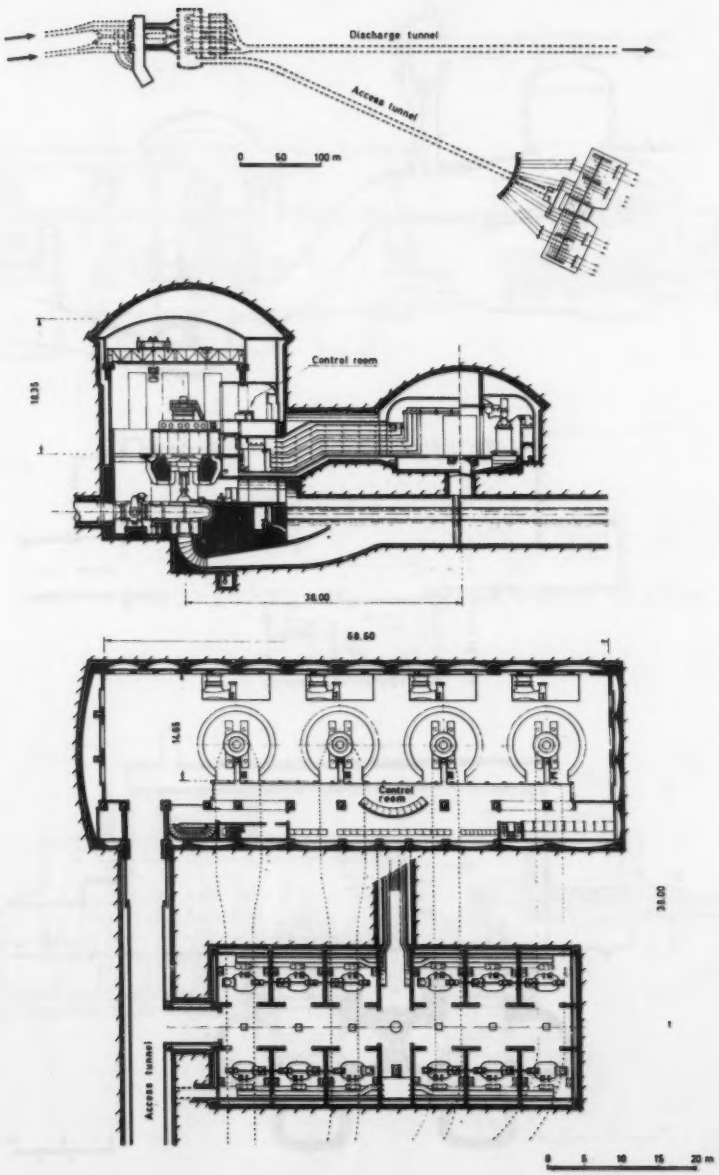


Fig. 8

ALA POWER STATION

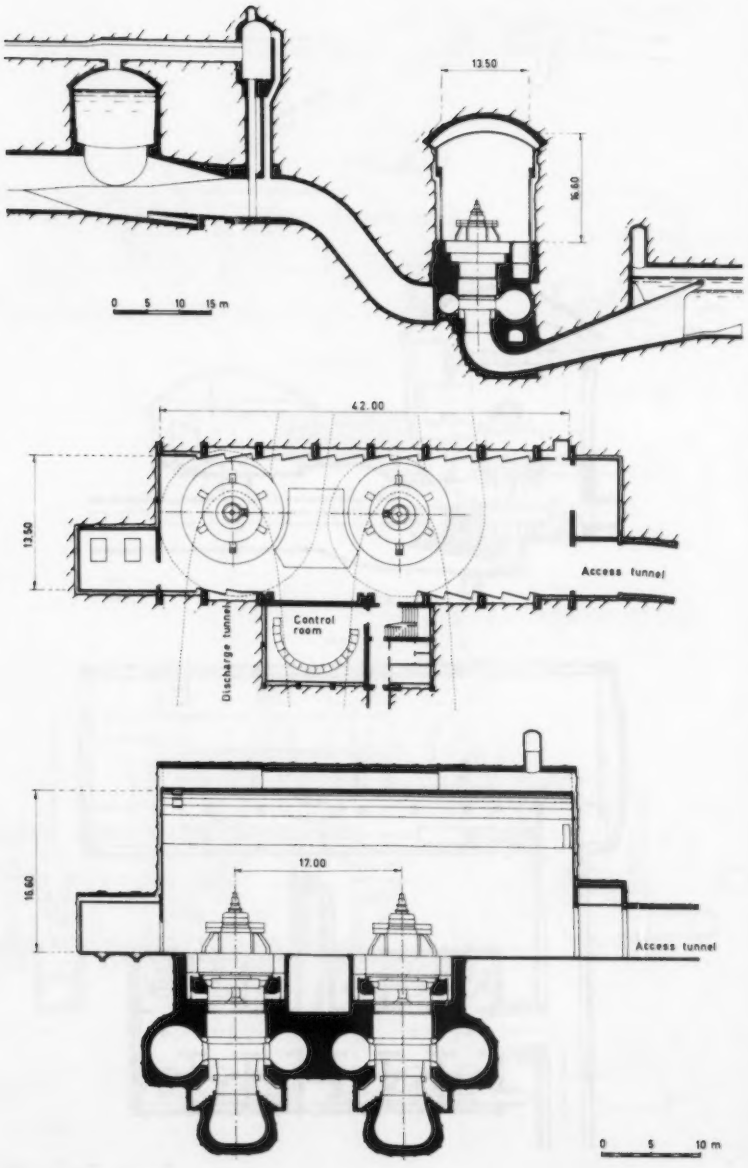


Fig. 9

AVISE POWER STATION

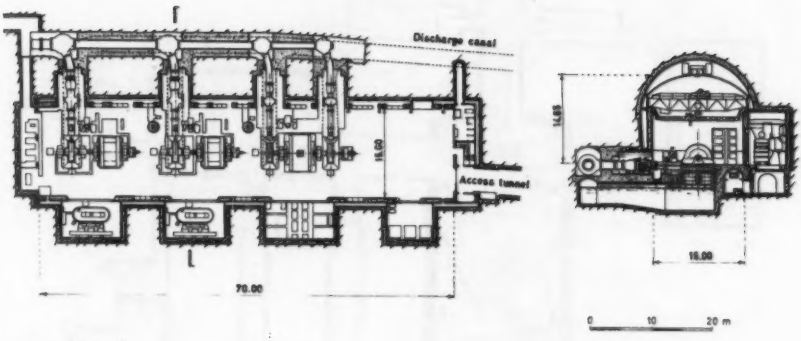


Fig. 1

ACRI POWER STATION

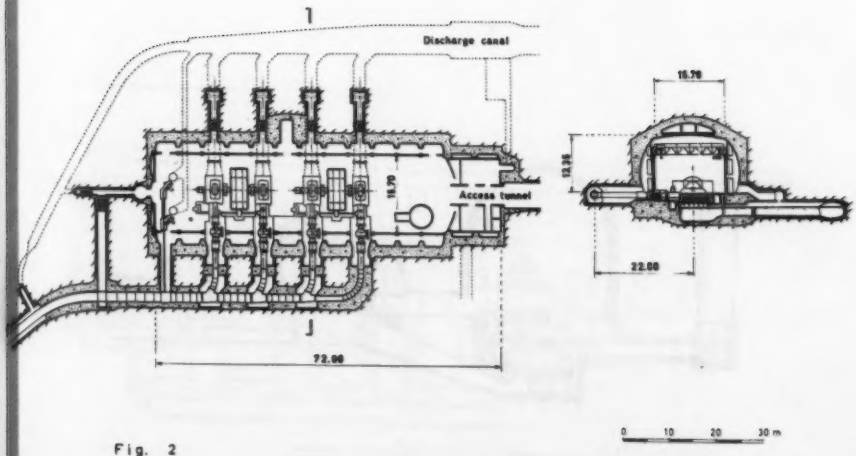


Fig. 2

Fig. 10

MONTORIO AL VOMANO POWER STATION

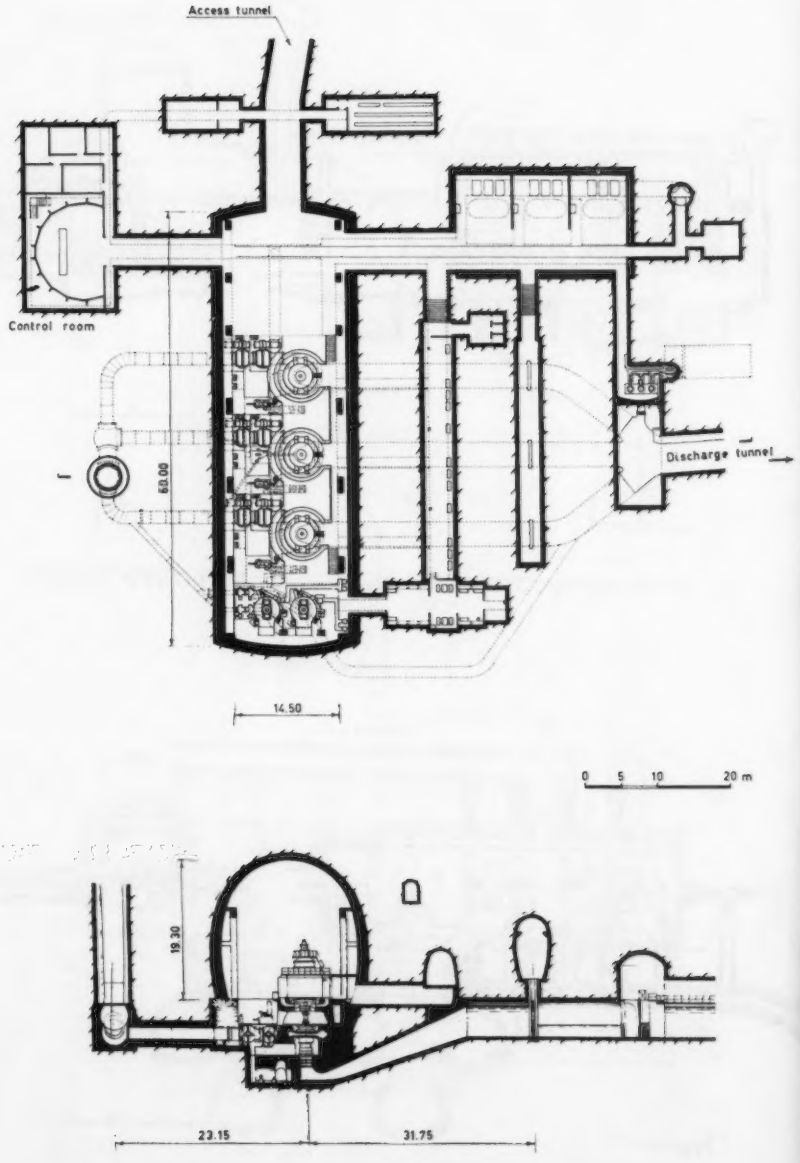


Fig. 11

PREMADIO POWER STATION

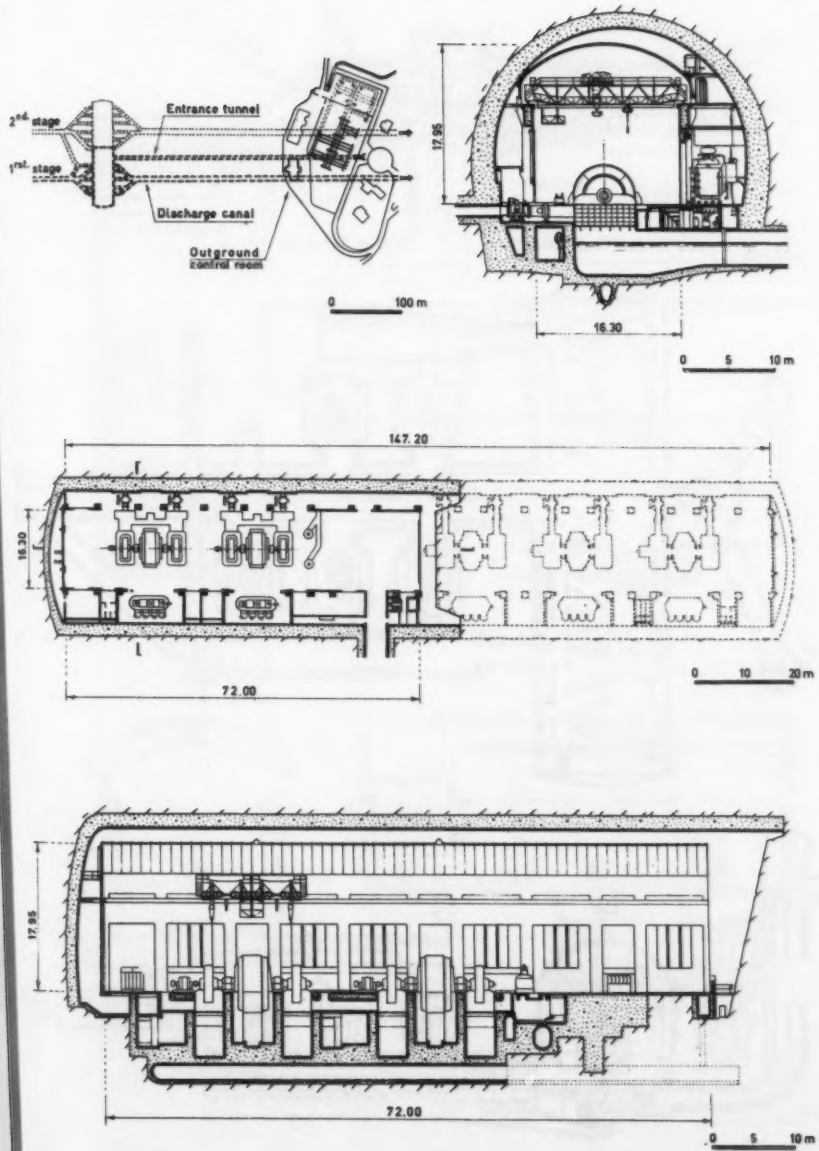
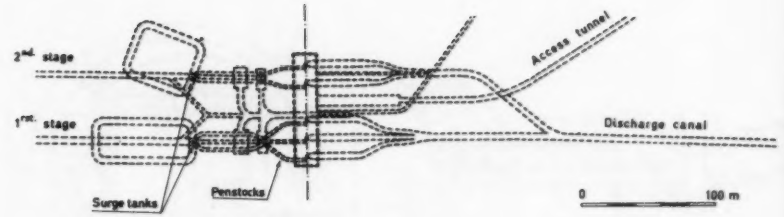
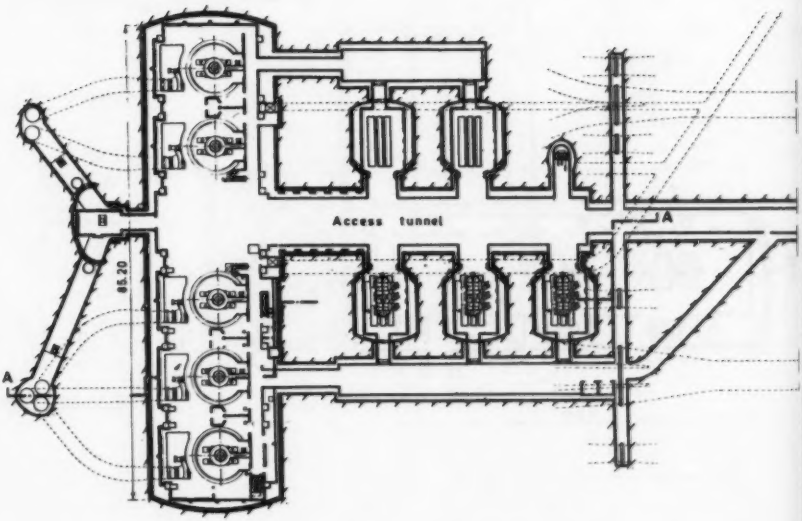


Fig. 12

SOMPLAGO POWER STATION





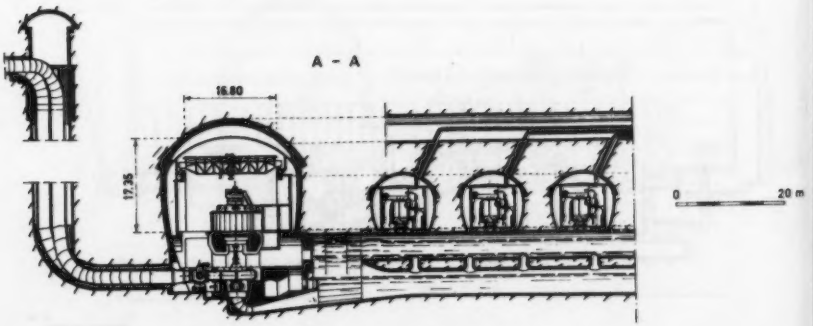


Fig. 13

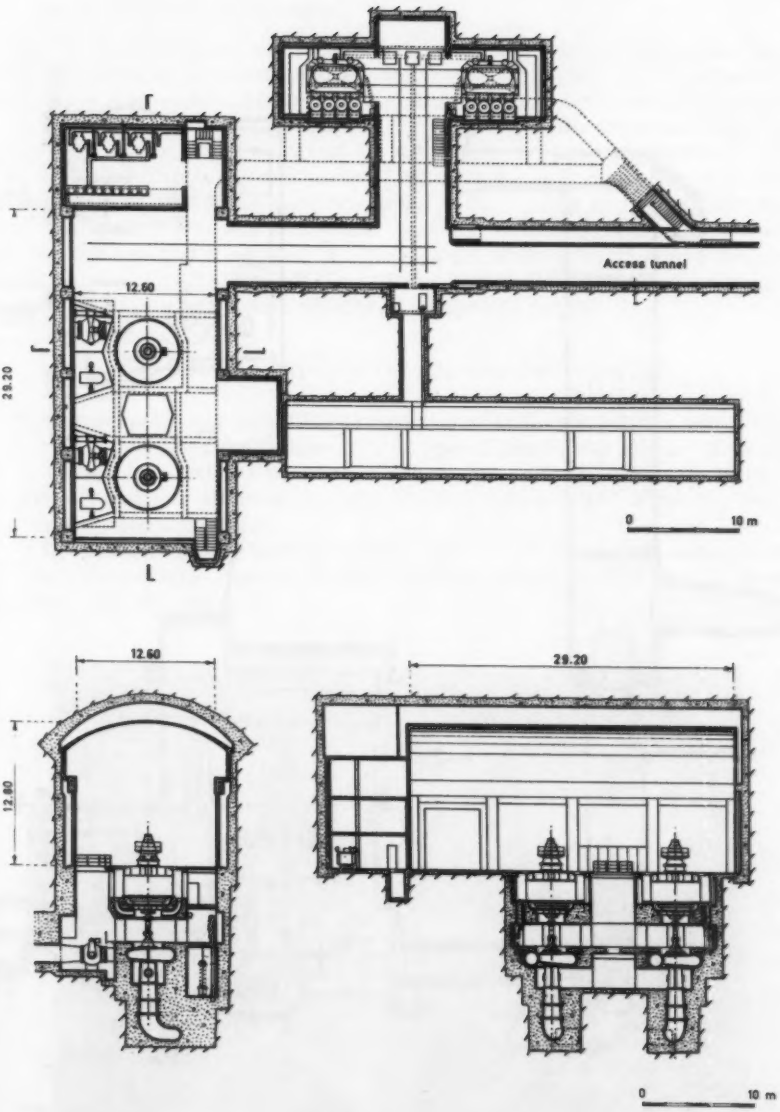


Fig. 14

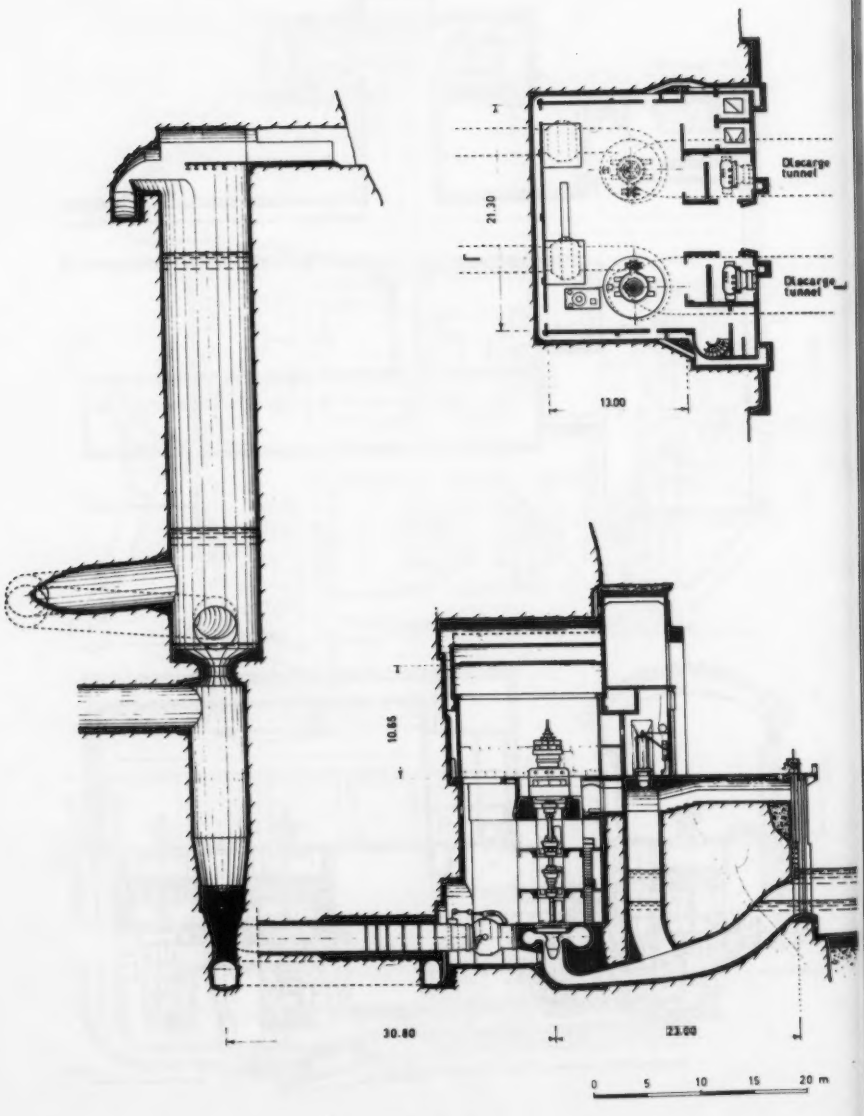

Discarge tunnel
21.30
Discarge tunnel
13.00
10.65
30.80
23.00
0 5 10 15 20 m

Fig. 15

Case B

(1) The sides of the valley are sufficiently high to allow the drilling of the supply tunnel at the dam foot elevation;
(2) The nature of the rock to be crossed by the outlet canal is such as to warn against the possibility of water infiltrations from the river along which the plant is developed.

When a first choice of the scheme has been made on the basis of the topographic conditions mentioned above, intermediate schemes can be adopted, considering also such conditions as the morphological and geological situation of the rock, the possibility of building the surge tank, the penstock shaft, accessory structures, road conditions, nearness to the transport network, etc.

In Italy topographic conditions are favorable to the construction according to Case B, in so far as the mountain sides of the valleys are generally rather high. The opposite occurs in Sweden. Namely, in Italy scheme A was used only at Coghinas and first head of Flumendosa; while scheme B was adopted in nearly all the remaining plants. Some power stations either cut away in the rock or in pits are to be considered as special cases.

Different Types of Transformer Locations

The conveniences of installing transformers underground has been investigated ever since the construction of this type of plants was begun. This solution answers at once the purpose of ensuring a uniform protection to all machinery against war offenses, of reducing ohmic losses and of raising the limit of electric stability.

At first transformers were generally placed outdoor or half-underground (Doblari, Cencenighe, Agordo, Stanga, Lumiei, Glorenza, Castelbello, Ponte Gardena, Ala, Bressanone).

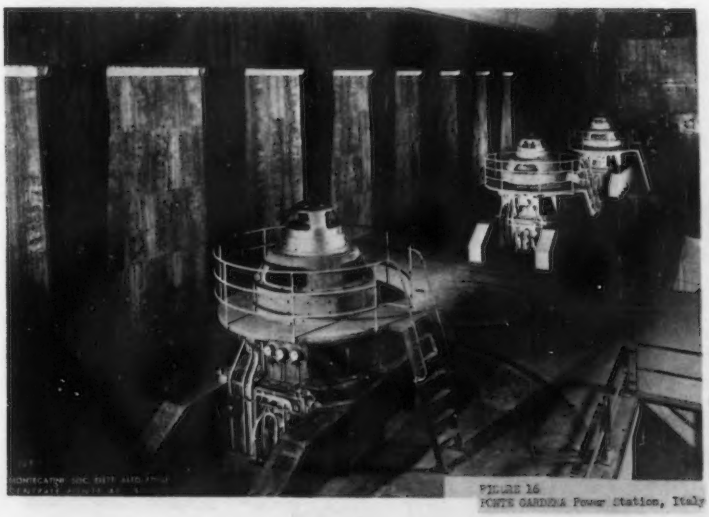


FIGURE 16
PONTE GARDENA Power Station, Italy

In recent years, however, the trend towards the underground solution has been accentuated. This solution, which was hindered in the past by the fact that very high-tension cables were not sufficiently tested and set up, presents now no technical difficulty, in so far as one-phase cables up to 250 kv and over have reached a high degree of perfection.

This underground solution is no doubt the most rational one. Yet it requires a particular economic evaluation for, although high-tension cables are costlier than bars, there are some favorable and important factors to be considered. First of all power losses during transmission from alternators to transformers are—as was previously mentioned—smaller. A bridge crane for lifting the transformers can be saved by using that of the generator room. In the economic evaluation, among the construction factors, the length, width and inclination of the access tunnel must of course be taken into account.

The economic evaluation carried out by designers in the various cases of Italian underground power stations has resulted in half the power stations in operation having underground transformers. Of the power stations under construction in late 1957, six have outdoor transformers and two have underground transformers.

With regard to the position of underground transformers in respect to the main room and access tunnel, four basic layouts may be adopted (Fig. 5).

(1) Transformers are placed inside one special excavation, which can be either connected to the access tunnel (Fig. 5) as at Soverzene (Fig. 8), Pontey, Acri, Grosio, Brunico; or independent of the access tunnel, with adit direct from the machine room (Fig. 5b) as at Montorio al Vomano (Fig. 11) Villa S. Maria, Stazzona, Grigno;

(2) Transformers are placed in separate excavations along the access tunnel (Fig. 5c) as at Somplago (Fig. 13) Plava, Morasco;

(3) Transformers are placed in the main excavation (Fig. 5d). In this case they may be set in special boxes at one end of the machine room, as at Belvisio, Malga Ciapela, Provvidenza, Regoledo, Lovero, or else along

FIGURE 17
DOBLARI Power Station, Jugoslavia
(formerly in Italy)

a wall of the machine room, within special boxes each corresponding to a unit, as at Premadio (Fig. 12), S. Massenza and Avise (Fig. 10).

Excavation Methods

The excavation technique of an underground power station evidently depends on the type of rock and the direction of its strata. It is therefore difficult to reduce it to definite schemes for—at least in our country—the nature of the rock on which the power stations are built is always varied and often difficult. We will try to give some ideas we generally follow.

Since there is a certain possibility of choosing the position of the large machine room, the main axis of the room is generally made to follow the direction most suitable for excavations considering the direction of the strata. Thus, the axis will preferably follow a direction at right angles with the intersection line between the plane of the strata and the horizontal plane: therefore its direction will be at right angles, with the strata themselves, if these are vertical. Should this be impossible on account of other structures (access tunnel, outlet tunnel, etc.) the direction of the axis is given a certain inclination in respect of the strata, but possibly not parallel to them.

At any rate this situation, besides being determined by preliminary geognostic investigations, is generally controlled by means of test tunnels (drilled, for instance, along the plane of the arched roof) which are used, later on, as cable outlets, emergency exits, aeration shafts, etc. We think that this control is very advisable in our geological conditions: it has sometimes led us to change the general axis of the structure or shift it to a different position (Ampezzo power station) and consequently to alter also, in some case, the trend of the access tunnel (Glorenza power station).

As a rule, excavations for a power station begin from the arched roof of the main room which in our country, on account of the types of rock to be

FIGURE 13
MUSCARONE Power Station, Italy

found, is almost always lined with concrete. There has been no case so far in which roof bolts could be used for fastening unstable rock strata to deep and firmer ones, leaving the rock of the arched roof unlined. Roof bolts have however been used for the rock of the sides. For instance at Brunico power station, where the rock (unsound, but not thrusting) was bolted immediately after excavation and gunited to avoid air exposure and consequent dangerous degradations.

When carrying out arched roof excavations—which have sometimes been made difficult by the width of the room to be excavated—we enter, so to say, into its mass by means of the tunnels which will later be used as access and discharge tunnels, so that we have three possible working planes: (1) arched roof plane; (2) machines plane; and (3) discharge plane. This enables us to choose with a certain liberty the excavation method best suited to the central core.

Thus, if the sides of the room are of such a nature that no rock sliding or thrust may be expected, as soon as the lining of the arched roof is completed, the central core is excavated by means of vertical mine volleys, the resulting material being loaded on trucks by excavators or loaders, and carried outside through the access and discharge tunnels. If the conditions of the rock require caution, the side zones are excavated by means of side passages and wells proceeding from planes 2 and 3; once the arched roof is reached, the unexcavated core is used for reacting against the rock thrust by means of struts. The extent of these side excavations is determined by the various situations; in some cases they can be reduced to almost the same dimensions as pillars, in general highly reinforced and encased in the concrete of the arched roof when the ground thrust is considerable (Acri power station). When these pillars are constructed, the side zones in between are excavated and lined, the lining being given an arched shape by vertical generatrixes.

When mechanical loading means and transport vehicles were not so widely used as at present, the semiautomatic loading of decauville wagons ensued

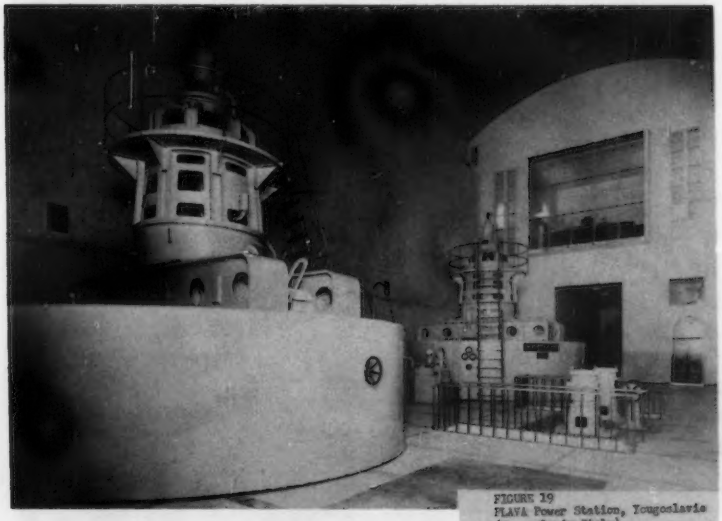


FIGURE 19
PLAVA Power Station, Yougoslavia
(formerly in Italy)

through the system of wells that connected the access and exit tunnels, with the three working planes mentioned above. By this solution the wells, which were generally dug on the axis of the machines, acted as "silos" of the excavated material and the central core was excavated by enlarging them further (Soverzene power station).

Cross-Section Shape and Lining Features

The different shape of the cross sections adopted depend obviously on the rock conditions.

At any rate, owing to the natural geological conditions in our country and the technical development of construction equipment, the range of excavation system adopted so far is considerably wide. Rock conditions, however, were never so favorable as, for instance, in Sweden, although some power stations were excavated on excellent rock (Doblari, Plava, Stanga, Lana and others).

The shape of the cross sections most widely used in Italy is the vertical side shape, mostly lined, with arched roof where in general the rock is substantially sound and not thrusting. Different types of oval and horseshoe cross sections have been used in thrusting or difficult rock.

Different construction devices resulting in various features of the cross-section profiles have been adopted where rock conditions were particularly poor or varying, or in order to face difficulties arising after the beginning of construction. For instance at Ampezzo, deep cuts were made into the rock abutments of the curved roof; into these cuts, reinforced extensions of the vertical pillars were introduced (see Fig. 7).

Most of the underground power stations in Italy present, as mentioned in the above, a concrete lining against the rock both at the arched roof and sides, the lining being simple or reinforced. The resulting machine room is in general isolated by means of hollow spaces obtained through an inside wall made

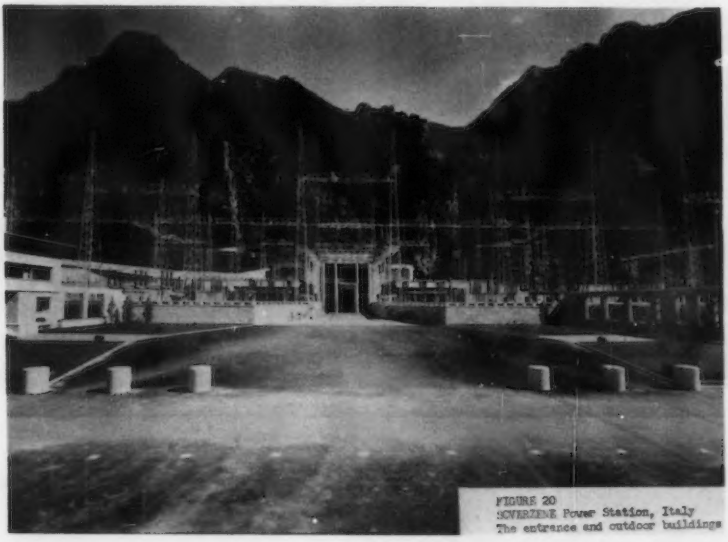


FIGURE 20
SOVERZENE Power Station, Italy
The entrance and outdoor buildings

FIGURE 21
SOVERZEN Power Station, Italy
Alternator Room

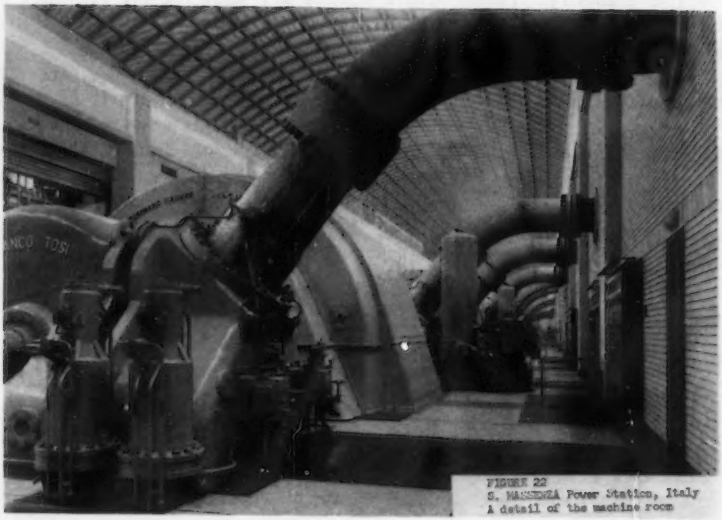


FIGURE 22
S. MARGHERITA Power Station, Italy
A detail of the machine room

FIGURE 23
MALGA GIAPELA Power Station, Italy

mostly of tile masonry. These hollow spaces are used for the drainage of natural water which—in spite of injections—often filters through the lining masonry against the rock.

Nevertheless, examples of power stations without inside walls, are also to be found, as at S. Giacomo, Monte Argento, S. Giustina, Montorio al Vomano, Narni; as well as examples of power stations without false-vault ceiling; as at Coghinas, Zappello, S. Giacomo, Montorio al Vomano, Quart. Lana power station, recently completed, has no lining against the rock of the sides—between the pillars—for a considerable part of its perimeter, although the roof is lined. (See photograph)

In Fig. 6 the underground power stations have been grouped according to their lining features and cross-section shape.

The Underground Power Stations of the S.A.D.E. and Montecatini Companies

Among the pioneering electrical groups that undertook and largely developed the construction of underground power stations in Italy are S.A.D.E., Societa Adriatica di Elettricita—Venice, and MONTECATINI, Societa Generale per l'Industria Meccanica e Chimica—Milan, to which the authors of this paper belong.

The activity of the companies in this field began in 1937 with the Ponte Gardena Power Station (1937-38) built by Montecatini, and the Cencenighe and Doblari power stations (1937-1939) built by S.A.D.E.

Ponte Gardena is equipped with 3 Francis turbines with a total capacity of 55,000 kw and flow rate of 100 m^3/s. Cencenighe was the first station of the Cordevole scheme, which now comprises three more underground power stations: Agordo (1938-40), La Stanga (1940-43), and the recently finished Malga Ciapela (1950-56) Station, with a total capacity of 106,000 kw. Doblari

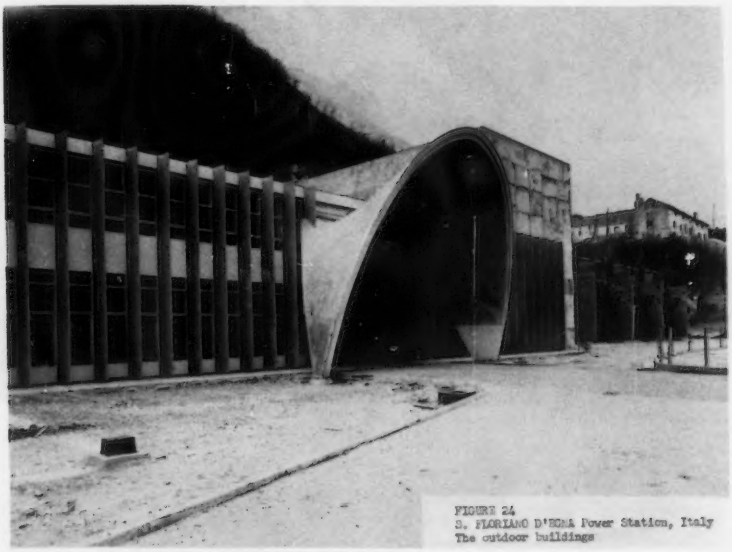


FIGURE 24
S. FLORIANO D'ENNA Power Station, Italy
The outdoor buildings

FIGURE 25
NEMBIA Power Station, Italy
Access shaft

on the Middle Insonzo is remarkable for its general characteristics, especially the hydraulic ones: the surge tank and the tail water tunnels. For that reason it may be actually considered as the first large underground station built by Italian engineers. Plava Power Station, the second one on the Isonzo, was built one year later. The total capacity of the two stations is 52,000 kw. Both are now in the Yugoslav Republic.

In the same years before the second world war, the Bressanone power station (1938-40) was built by Montecatini for the State Railways; it may be regarded, from the point of view of the power installed, as the first large power station built by two companies. Fed by a concrete penstock 6 m in diameter, 350 m long, Bressanone has a total installed capacity of 142,500 kw; the maximum head is 164 m, the maximum flow rate is 72 m^3/s.

During the war, the construction works of the two high-head Ampezzo (Fig. 7a) and Glorenza (Fig. 7b) stations were started; their operation began in 1948. Ampezzo is equipped with three horizontal-shaft generating sets, each operated by a pair of double-injector Pelton wheels; maximum head is 470 m, total capacity 60,000 kw. Glorenza is equipped with two Pelton sets, in a layout similar to that of Ampezzo; maximum head is 590 m, total capacity 100,000 kw. Approximately in the same period Castelbello Station was built and equipped with 3 Francis turbines with a total capacity of 85,000 kw; maximum head is 290 m, maximum flow rate 30 m^3/s.

In 1946 the first works for the large Soverzene (Fig. 8) Station began, which was completed in 1951. Soverzene has until now the largest installed capacity of all the stations built by the two companies. It is fed by a complex system of pressure tunnels over 40 km long which convey water to two couples of prestressed concrete penstocks, and is equipped with 4 Francis turbines for a total capacity of 220,000 kw. The maximum flow rate is 88 m^3/s; maximum head 284 m.

From 1950 to 1953 the Construction Department of Montecatini built the Ala power station (Fig. 9), a low head station jointly operated by the same

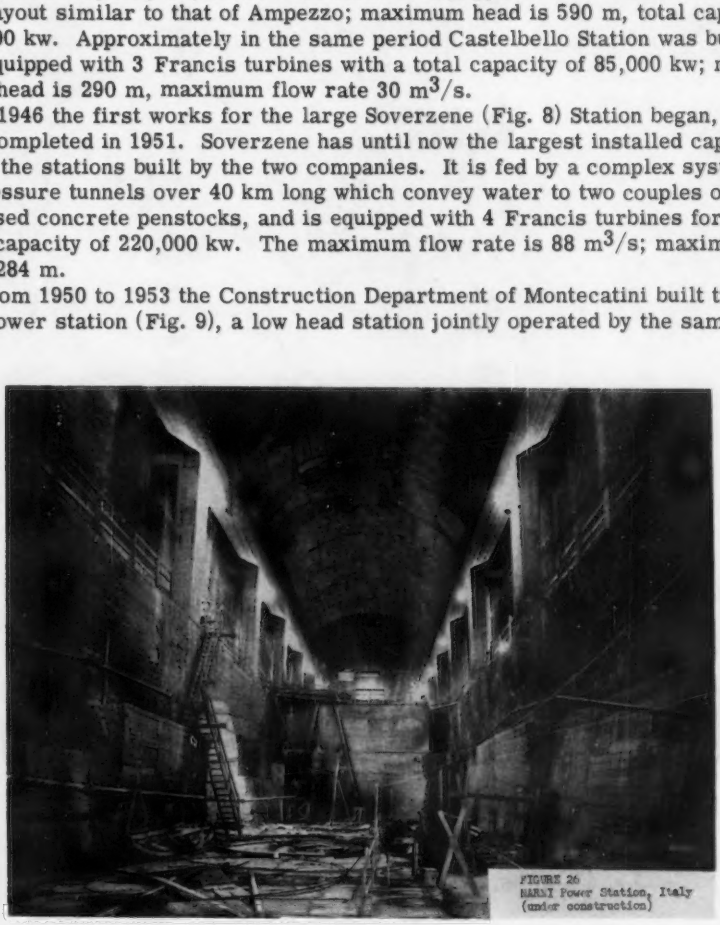


FIGURE 26
NARNI Power Station, Italy
(under construction)

company with the Selt-Valdarno Company. It is equipped with 2 Kaplan turbines each fed by a flow rate of 100 m^3/s; total capacity of generators is 45,000 kw.

In the years from 1952 to 1955 the two power stations of Gardona and Barcis were built. Gardona is a subsidiary station of the large Piave-Boite-Vajont scheme, the main station of it being Soverzene. Barcis (Fig. 15) is the upper plant of a series of existing power stations along the Cellina River, on which a further series of at least two more stations will be built in the near future. Barcis station is the only "cutaway" power station of the two companies; equipped with a long axis (20 m) 10,000 kw Kaplan set, operating under a head varying between 60 and 28 m. Another set will be installed. Maximum flow rate will be 45 m^3/s, capacity envisaged 20,000 kw.

The new Somplago power station (Fig. 13), which is being finished, will surpass Soverzene with regard to the capacity, thus becoming the biggest power station of the two companies. Somplago will be equipped with 5 Francis turbines, for a total capacity of 275,000 kw, two of which are already installed and under operation; maximum head is 290 m, maximum flow rate 110 m^3/s.

To sum up, of the 57 Italian underground power stations in operation, 17 were designed and built by the two groups—11 by S.A.D.E., 6 by Montecatini; 12 of them are at present in operation by the same companies, 1 jointly by Montecatini and the Selt-Valdarno companies.

The capacity installed at the date of June 1957 in the 17 power stations built by the two groups is 1,050,000 kw as against a total capacity of 3,167,000 kw installed at the same date in Italian underground power stations, which corresponds to 29% of the total. Besides, a further capacity of 175,000 kw is being installed in two of the same power stations of the two companies.

In addition to that, the two companies are now building two more underground power stations for a total of 31,000 kw. Of these the Brunico power station (Fig. 14) will be the larger. It will have 2 Francis sets with an installed capacity of 21,000 kw; maximum head will be 201 m, flow rate 22 m^3/s.

FIGURE 27
LOVERO Power Station, Italy
Exterior view

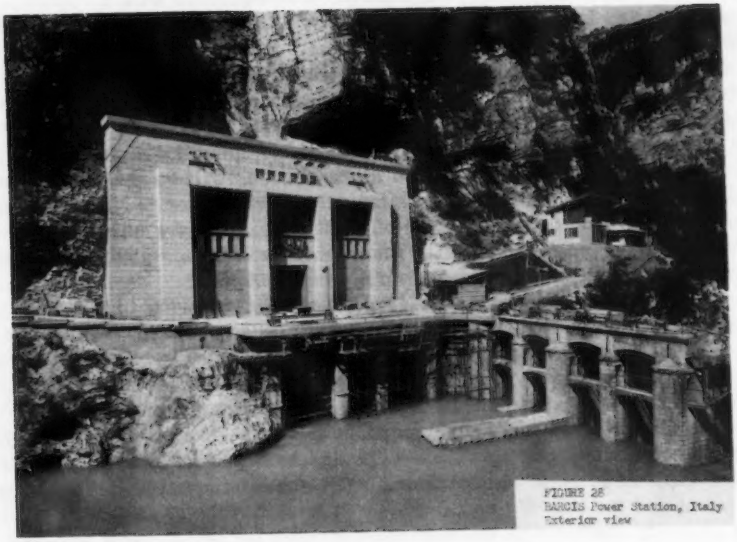


FIGURE 28
BARCIS Power Station, Italy
Exterior view

FIGURE 29
CUTILIA Power Station, Italy

Plans for 15 more stations with a total capacity of about 500,000 kw have been also designed.

The enclosed list reports data and characteristics of the power stations built and under construction by the two companies, as well as of the other Italian power stations.

Some of the stations of the two companies required special construction work to overcome ground difficulties (for instance: Ponte Gardena, Cencenighe and Agordo, Ampezzo, Somplago), for which reference should be made to the special technical literature. In the masonry works of Ampezzo and Somplago, special reinforcement devices had to be adopted, as explained in the previous chapters of the paper.

The main layout and dimension features are shown in the drawings attached. Some characteristics of underground plants and their surface structures are visible in Figs. 16 through 29.

ACKNOWLEDGMENTS

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Proceedings of the American Society of Civil Engineers

CONTAINMENT OF NUCLEAR PLANTS

Richard N. Bergstron,[1] F. ASCE

SYNOPSIS

Present practices in design and construction of containment vessels are reviewed. Consideration of types of energy available to produce internal pressures and resulting design pressures are discussed together with applicable codes for design. Effects of the shape and size of the containment vessel upon economies of the plant layout are discussed.

INTRODUCTION

The advent of the nuclear power plant has brought with it a new type of power plant building - the containment vessel. With this large enclosure which envelopes all or part of the nuclear plant has come a host of new problems in design and construction.

The nuclear reactors being constructed today have many safeguards built into the operating systems to protect against an accidental discharge of fission products which could cause radiation damage over a large area. For the reactors built in this country, consideration has always been given to the possibility that these safeguards might fail permitting the release of large amounts of contaminated gases and particulate materials. For the first experimental reactors, this problem was solved by building, operating and testing the reactor in remote areas where few people would be endangered. With the attempt to develop power reactors into practical components for commercial power plants has come the need to build these plants closer to large centers of population with the accompanying decision to build a gastight vessel enveloping portions of the nuclear plant in order to provide a last barrier to prevent the release of contaminated gases to the atmosphere.

In the United States, it has become accepted practice to build this gastight envelope as a pressurized steel containment vessel and therefore the

Note: Discussion open until May 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2285 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 6, December, 1959.

1. Associate, and Structural Engr., Sargent & Lundy, Chicago, Ill.

discussion in this paper will be concerned primarily with such vessels. The practices in the United States may however be due in part to the types or reactors which have been designed in this country. The gas-cooled reactor which has been popular in England which, over there, has escaped the burden of containment; has been authorized for construction in the United States also but it is not known at this time if a containment vessel will be required for this installation. The discussion in this paper will be based primarily on vessels for water-cooled, water-moderated reactors. Some nuclear scientists are of the opinion, however, that containment will be required as long as fission is used as the source of power.

Forces to be Contained

The fission products or other radioactive materials which must be kept from reaching the population are dangerous but would present no serious problem to the general population if they were merely released without being accompanied by some form of energy tending to drive them into the atmosphere. With no driving force acting upon them, they could be easily contained in a simple building.

There exist, however, several sources of potential energy in the form of stored thermal energy in the coolant and moderator and in potential nuclear releases or chemical reactions between the materials present. Theoretically, the total potential energy in these several sources is fantastically high. It is not known what part of this total available energy could be transformed to perform actual work but it is generally recognized that for the cases of nuclear releases and chemical reactions the values will be far below the theoretical. There is also a time lag between the periods at which these various types of energy become available although little experimental data is available to establish the magnitude of the time delay.

Release of the stored thermal energy in the coolant or moderator is easy to visualize since all this would take is a rupture in the reactor pressure vessel or piping system allowing the pressurized coolant to escape. Depending upon the thermodynamic properties of the coolant this can then flash to vapor giving up part of its energy to the air and producing a rapid increase in pressure inside of the containment shell. This is a very direct action and it is logical to predict that a large percentage of stored thermal energy can be transformed into pressure against the containment shell although all of it cannot be. The actions of escape and vaporization and expansion are not instantaneous and there will be some period of time required. A considerable amount of heat will be absorbed by cold surfaces in the vessel such as concrete walls and floors. Nevertheless, this release of energy represents the greatest portion of the energy that is expected to be released.

As a result it has become a fairly common practice in containment vessel design in this country to compute the increase in pressure due to a theoretical instantaneous release of the stored thermal energy in the coolant or moderator with no reduction for heat transfer to other surfaces and then study the effect of combining this with various percentages of the nuclear or chemical reactions. It is generally assumed that the main nuclear release, if any, will breach the reactor vessel and surrounding structure but the containment shell will not experience any shock wave from either this or explosive combustion in the air. Generally, it is felt that a design at normal allowable stresses for the pressure due to stored thermal energy with no reduction for heat transfer

is quite conservative and the possibility of a combination of full theoretical values for energy from several sources is too remote to be considered at normal design stresses.

An example of a comparison between pressures due to various sources of energy is shown in Table I. This shows a tabulation of the theoretical pressures for a nuclear plant in which various combinations are made of various chemical reactions with the case of a rupture of the primary system resulting in a release of the coolant. In this case it was decided to design the containment vessel for 21 psig and follow the code in testing the vessel to 26.25 psig. This was based on the discussions in preceding paragraphs considering the fact that there will be heat transfer to other surfaces with a corresponding reduction in heat energy available for expansion of the gas and hence the pressure due to stored thermal energy will be considerably less than 20.3 psig. The difference would be available to take care of whatever energy was contributed coincidentally from random sources. At worst, it can be seen that the most pessimistic figures still lie within the range of an overstress that is not unreasonable.

Following this line of thinking it has commonly been assumed that design for containment vessels will be conservative if it considers all of the energy from an instantaneous expansion of the high-pressure, high-temperature water from the reactor system into the containment volume as a constant internal

TABLE I. COMPARISON OF CONTAINMENT VESSEL PRESSURES FOR ONE NUCLEAR PLANT WHICH MIGHT RESULT FROM VARIOUS SOURCES. (Not Including Nuclear Energy Releases)

Case	Accident	Initial Air Temp. Of	Containment Pressure psig
1.	Primary System Rupture	70	20.3
		100	19.3
2.	Primary System Rupture Plus Chemical Reaction of 25% of ZR With Water	70	22.6
		100	21.6
3.	Case 2 Plus Hydrogen Burning	70	24.4
		100	23.4
4.	Primary System Rupture Plus One Secondary Heat Exchanger Rupture	70	26.5
		100	25.5
5.	Case 4 Plus 25% ZR - Water Reaction	70	28.7
		100	27.8
6.	Case 5 Plus Hydrogen Burning	70	30.7
		100	29.6

energy mixing process in a closed thermodynamic system. The first law of thermodynamics and the Gibbs-Dalton law for gas mixtures are assumed to apply. Assuming that the contained air in the vessel is dry, the following equations, shown on pages 7 and 8, must be satisfied;

(1)

$$P = P_s + P_A \left(\frac{460 + T_2}{460 + T_1} \right)$$

where P = final absolute mixture pressure - psia
Ps = saturated steam pressure at T_2 - psia
P_A = initial air pressure - psia
T_1 = initial air temperature - °F
T_2 = equilibrium mixture temperature - °F

(2)

$$W_{a1}\left(U_{a1} + \frac{V_{a1}^2}{2gj} + Z_{a1}\right) + W_w\left(U_{f1} + \frac{V_{w1}^2}{2gj} + Z_{w1}\right) =$$

$$W_{a2}\left(U_{a2} + \frac{V_{a2}^2}{2gj} + Z_{a2}\right) + (1-x)\,W_{w1}\left(U_{f2} + \frac{V_{w2}^2}{2gj} + Z_{w2}\right) +$$

$$x\,W_{w1}\left(U_{g2} + \frac{V_{w2}^2}{2gj} + Z_{w2}\right)$$

Wa = weight of air - lbs.
Ww = weight of water - lbs.
Va = velocity of air - fps.
Vw = velocity of water - fps.

Ua = internal energy of air Btu/lb.
Uf = internal energy of water Btu/lb.
Ug = internal energy of steam Btu/lb.

g = gravitational constant 32.2 ft/sec^2
j = heat equivalent to work 778 ft lb/Btu.
z = potential energy due to position

x = steam quality (percent)

Subscript 1 = initial condition.
Subscript 2 = final condition.

In most cases of practical interest the potential energy terms z and the kinetic energy terms $\frac{V^2}{2gj}$ may be neglected. Thus equation (2) reduces to

(3)

$$W_{a1}\,U_{a1} + W_{w1}\,U_{f1} = W_{a2}\,U_{a2} + (1-x)\,W_{w1}\,U_{f2} + x\,W_{w1}\,U_{g2}$$

These equations can be solved by trial and error to show the relationship between the number of cubic feet of free containment vessel volume which is required for each pound of hot water and steam contained in the reactor system and the equilibrium containment vessel pressure. Such a relationship is shown in Fig. 1 for several curves corresponding to different pressure conditions in the pressurized water or steam system of a boiling water reactor.(1) For a pressurized water reactor the curves used should correspond to the

average saturation temperature in the system. The abcissa indicates the pressure resulting in the shell after complete flashing to the volume shown on the ordinate. It is interesting to note how rapidly the volume must increase as the pressure is reduced. This is a factor which possibly has not been given sufficient attention in the design of containment vessels to date and will be discussed further in the following paragraphs.

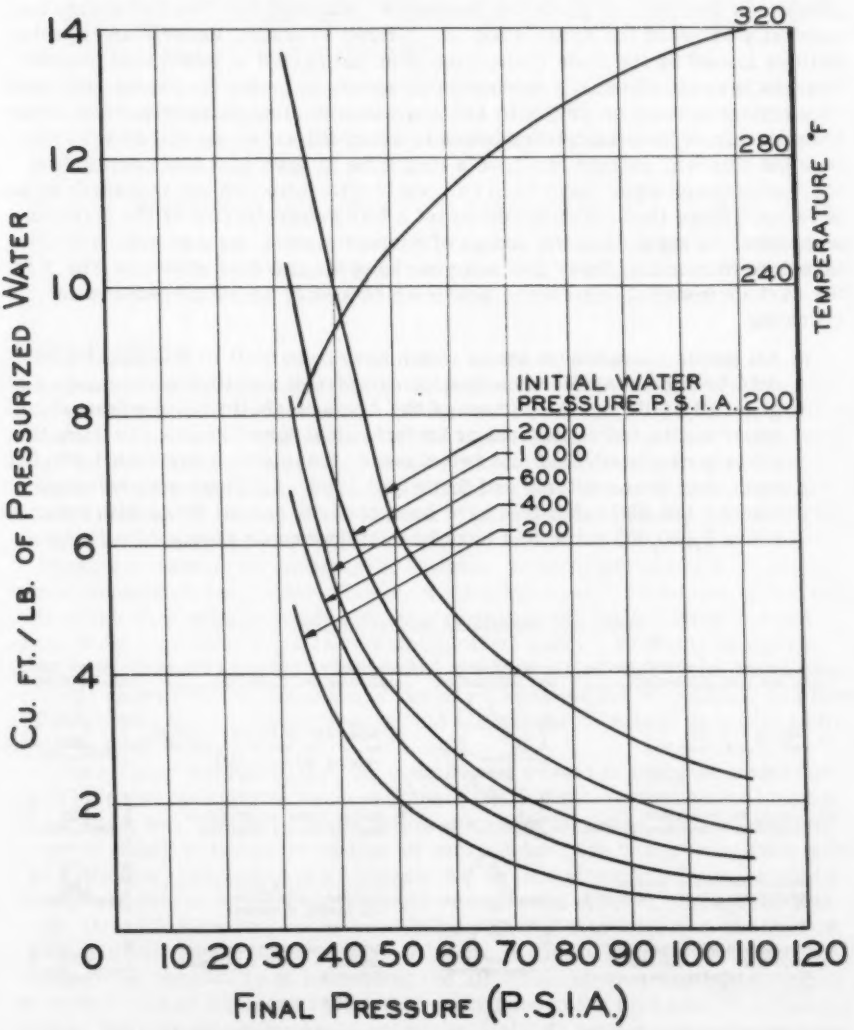


FIG. 1 CONTAINMENT SHELL VOLUME REQUIREMENTS

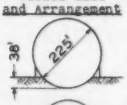
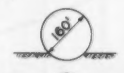
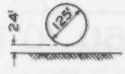
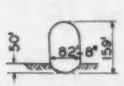
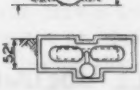
Review of Present Installations

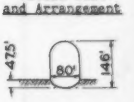
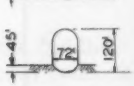
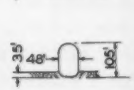
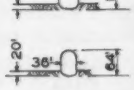
Before studying the various other factors which determine containment vessel arrangement and design, it is of interest to review some of the installations which have been built or are being built in this country. Such a comparison is shown in Fig. 2.

In examining the arrangements discussed in Fig. 2, it should be borne in mind that these were developed with no specific set of ground rules for design conditions available to guide the designers. Although the detailed design has generally followed the ASME Code for Unfired Pressure Vessels and special rulings issued by the Code Committee over the period in which containment vessels have been built. A movement is underway by the American Standards Association sponsored jointly by the American Nuclear Society and the American Society of Mechanical Engineers to establish safety standards for reactors but this will undoubtedly take a long time to develop. It is certain that the installations which have been completed will influence the standards to be developed since they actually represent a fair cross-section of the opinions of people now engaged in the design of nuclear plants. As a result, it is interesting to note that there are many striking similarities shown in Fig. 2 for the various installations. Some points which should be mentioned are the following.

1. All major containment shells which have been built in this country to date have been either spherical or cylindrical. Cylindrical vessels have generally followed the pattern of the Argonne shell with hemispherical upper heads and ellipsoidal or torispherical lower heads. To date, the vessels can be divided into two classes; those with more than 1,000,000 cubic feet gross volume and those with less. All those with volumes above 1,000,000 cubic feet have been spheres and all those with volumes below 1,000,000 cubic feet have been cylinders or groups of cylinders.

FIGURE 2A - PROPERTIES OF AMERICAN CONTAINMENT VESSELS

Type, Nuclear Electrical Rating and Location of Reactor	Vessel Size and Arrangement
1. Testing Facility West Milton, New York	
2. Boiling Water Reactor 180,000 kw - Dresden, Illinois	
3. Pressurized Water Reactor 163,000 kw - Indian Point, N.Y.	
4. Pressurized water Reactor 134,000 kw - Yankee Atomic Electric Company, Rowe, Massachusetts	
5. Air Force Nuclear Engineering Test Reactor, Wright-Patterson Air Force Base, Dayton, Ohio	
6. Pressurized Water Reactor - 60,000 kw - Shippingport, Pennsylvania	

Type, Nuclear Electrical Rating and Location of Reactor	Vessel Size and Arrangement
7. Experimental Breeder Reactor No. 2 - 20,000 kw - Reactor Testing Station, Idaho	
8. Enrico Fermi - 100,000 kw Lagoona Beach, Michigan	
9. Indirect Cycle Boiling Water Reactor - 22,000 kw - Elk River, Minnesota	
10. Experimental Boiling Water Reactor - 5,000 kw - Lemont, Illinois	
11. Developmental Boiling Water Reactor 5,000 kw - Vallecitos Atomic Laboratory, Pleasanton, California	
12. Army Package Power Reactor No. 1 - 1,855 kw - Ft. Belvoir, Virginia	

2. Design pressures have averaged between 20 and 30 psig with two cases lower at approximately 15 psig and three above at 45 to 65 psig.

3. Specifications for material have been quite uniform with all but one case using A 201 steel conforming to the A 300 specifications for heat treating and Charpy testing.

4. Plate thickness for the main portions of the shells have been kept below 1.5 inches thus relieving them of any need for field stress-relieving the main shell as required by the ASME Code and Code Case 1226-1 for plates over 1.5 inches in thickness.

5. Practically all privately owned containment vessels will carry the code stamp showing that they conform to the ASME Code for Unfired Pressure Vessels. In general, this is done because many state laws could be interpreted as requiring it.

6. Most vessels have been subjected to radiographing of 100% of the welds.

7. Specified leakage rates for 24 hours have varied from 0.1% to 1.0% of the volume. In actual quantities of gas released, the leakage rate varied from 635 cubic feet to 30,000 cubic feet at various pressures.

Design Practices

In general containment vessels built to date have been designed in accordance with Section VIII of the ASME Boiler and Pressure Vessel Code, Unfired Pressure Vessels and applicable case interpretations by the Code Committee. This was a logical step since this is a well known and widely accepted code which over a period of many years has been written to cover a large variety of vessel types. It is a code recognized by many state laws and one which fabricators and designers are familiar with.

There are certain disadvantages, however, in applying this code to the design of containment vessels. The application has been a direct one to the main body of the code with only a few special rulings to set containment vessels apart in any way from vessels containing oil or water. Probably one of the most important of these was ruling 1224-1 which stated that containment vessels fall under the classification of vessels containing lethal contents and hence are subject to all the restrictions of that classification except as modified by other special rulings.

This means, therefore, that the containment vessel is designed under the same rules which apply to vessels subject to frequent or continuous loading. As a result, this design is based on low allowable stresses, with a generous factor of safety in the order of four or more, which are based on vessels subject to fatigue, creep or embrittlement due to cold working. These, however, are not conditions which a containment vessel must endure. Except for test loads, the major loading on the containment vessel may never be attained or if attained will be encountered very seldom. It seems logical, therefore, to consider the possibility of increasing the allowable stresses and thus reduce the cost of one of the many things that contribute to the high cost of nuclear plants. Such an increase should not be inordinately high since the consequences of a containment vessel failure would be extremely serious but a reduction in the factor of safety to 3 to 1 might be justifiable.

In considering a reduction in the factor of safety, careful consideration must be given to the fact that formulae given in the code are approximations

FIGURE 2B - PROPERTIES OF AMERICAN CONTAINMENT VESSELS

Vessel No. (See Fig.2A)	Containment Vessel Gross Volume Cu.Ft.	Radiograph	Design Pressure PSIG Internal/ External	Plate Thickness and Material	Code Stamp	Year Built	Specified Leakage Rate for 24 Hours
1.	6,000,000	100%	20 - 0.8	0.9" - 1.1" A201 - A300	No	1953	0.5% *
2.	3,600,000	100%	29.5 - 1	1.25" - 1.4" A201 - A300	Yes	1958	0.5% *
3.	2,140,000	100%	25 - 1.25	0.89" - 1.03" A201 - A300	Yes	Under Const.	1.0% *
4.	1,020,000	100%	31.5	0.875" - 1.25" A201 - A300	Yes	Under Const.	1.0% at 15 psig
5.	744,000	100%	16 (test)	0.563" - 1" A201 - A300	No	1957	0.1%
6.	635,000	100%	52.8	1"-125",0.66"-1.25", 0.75"-1" A201 - A300	Yes	1956	0.10% *
7.	approximately 584,000	About 10%	24 - 1	1" cyl. wall A201	Code Design	Under Const.	0.2%
8.	415,800	100%	32 - 2	1.03" - 1.25" A201 - A300	Yes	1957	0.11%
9.	415,000	100%	21 - 0.5	0.5" - 1.125" A201 - A300	Yes	1959	0.1%
10.	400,000	About 10%	15 - 0.5	0.375" - 0.625" A201 - A300	No	1955	0.25%
11.	approximately 160,000		45	0.436" - 0.875" SA212	-	1956	1.0% *
12.	52,700		65	0.875" shell	-	1956	-

*Leakage percentage at design pressure or as indicated.

and the code covers primarily vessels in which the openings through the shell are small and are treated as small holes in an extremely large plate. The designer must recognize these facts and realize that allowable stresses which are ordinarily considered conservative do not offer sufficient protection in such cases without a special analysis.

A point which is recognized but not commonly emphasized, is the fact that while the major portion of the shell is subjected to membrane stresses only, certain areas such as the junction of a dome head to a cylinder are subject to bending moments of major importance. The stresses at the junction of a cylinder and its head may be several times the stress in the main portion of the cylinder, but while the practices prescribed by the present code recognize the existence of these high localized stresses due to discontinuity they call for the same thickness of plate as is used throughout the remainder of the shell. Basically this means that either the steel at this point is highly overstressed or else that the allowable stresses have been set so low that the steel in the other portions of the shell are not working to full advantage.

There is a definite need to recognize the fact that containment vessels are a type of structure which is different from the type normally considered under the code. As outlined previously, consideration should be given to the fact that design loads will be rarely attained and a higher allowable stress permitted for the areas of the shell which are subject to membrane stresses only. Higher stresses may also be permitted in the areas subject to higher localized stress concentrations, if based on actual stress conditions but the use of these should be tempered by the accuracy to which these localized stresses can be determined. It appears that the problem of determining localized stresses at the juncture of the dome to the cylinder has been made much simpler by recent papers such as ASCE 2858 by Huddleson and Salvadori, in which they outline approximations which may be used for these calculations. If similar investigations can be made on the stress distribution around large openings in cylindrical and spherical vessels and incorporated in the code, the use of a higher allowable design stress seems quite logical.

There are probably numerous advantages to keeping the code regulation of containment vessels as a part of the ASME Pressure Vessel Code since this can be incorporated as a revision of a code which is widely recognized in most states and as such would not encounter the confusion and delay to which an entirely new code would be subjected. It does seem, however, that the special problems of a containment vessel are numerous enough to warrant the creation of a separate section of this code rather than trying to define them by special rulings permitting deviations from portions of the code. The ASME, American Nuclear Society and the American Standards Association are now working to create standard procedures. Considerable confusion and expense can be eliminated if a proper criteria for determining design conditions can be established. It certainly seems logical to argue that all containment vessels built in this country should conform to the same criteria except for whatever variation may be logical between populated and unpopulated areas.

*Leakage percentage at design pressure or as indicated.

Construction Practices

Construction practices on most of the containment shells built to date have been similar and have been generally arranged to produce an erected steel shell complete on steel columns holding the bottom of the vessel clear of the foundation or sub-steel in order to provide access to all points of the vessel.

This has been done to conform with the provisions of the code with regard to testing of the vessel. This is generally meant that erection starts with the placing of steel columns joined together with a complete belt of plates as shown in Fig. 3. Once the belt course is completed erection follows above and below the belt. Field welding follows as each course is erected and much of it is done by automatic welding machines.

Procedures for testing the adequacy of construction have a great effect upon construction operations. It is common practice to specify that joints in the plates shall be double butt welds and shall be accessible from both sides during testing. The vessel is then completed, pressurized to some percentage of the design pressure, and all joints tested for leakage by soap bubble or appropriate tests. After this test is completed, the pneumatic test is applied as prescribed in the Unfired Pressure Vessel Code to bring the internal pressure up to 1.25 times the design pressure. After the structure proves its ability to withstand this pressure, the pressure is reduced to the design pressure and held at this point. Attempts are then made to measure the leakage within limits at approximately 0.1% of the contained volume per day. The code requirements for such vessels call for all welds to be radiographed.

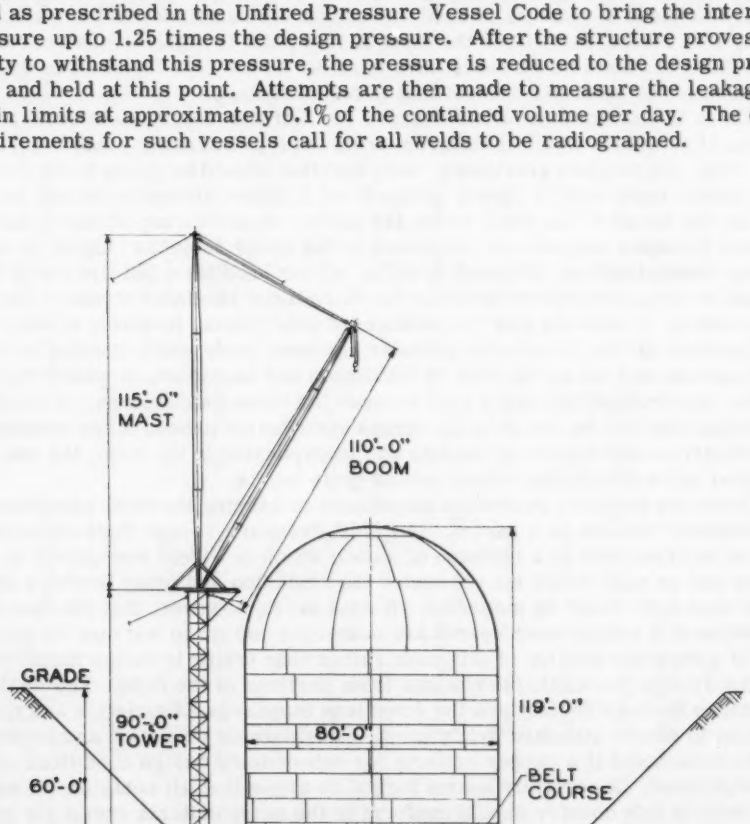


FIG. 3 ERECTION OF EBWR SHELL

This procedure means that both sides of the entire surface of the shell shall be accessible and necessitates leaving an inspection space below the bottom of the shell, requiring temporary supports of the shell during construction and inspection operations. Following completion of the inspection and testing, the space below the shell is usually filled in some manner to transmit the structural and equipment loads to the soil or foundation below. A large construction opening is then cut through the side of the shell and all interior construction operations are handled through this opening. This increases the difficulty of concreting operations within the shell and the erection of equipment inside the shell, as all material and equipment must be brought through this opening and, in many cases, the use of cranes in the interior is hampered seriously. The added cost of these difficulties is difficult to estimate for either the engineer or the contractor, but they are obviously a major cause of high construction costs in nuclear plants.

A recent ruling of the ASME Boiler and Pressure Vessel Code relative to nuclear installation points out a method where this difficulty can be reduced. This is found in Case No. 1228 on which a special ruling was made in December, 1956, to cover the case of two-stage construction of a containment vessel. This ruling provides that the lower portion of a containment vessel can be constructed and covered so that the welds will not be available for inspection at a later testing date provided that certain conditions be met. These conditions are that the bottom be constructed without openings or penetrations; all welds be double butt welded, radiographed, and fully inspected and tested for leaktightness by means such as Halide leak detector tests. This would mean that the lower portion of the shell could be completed with double butt welds, inspected, and then either lowered into a prepared footing or the space below filled in.

The construction of the internal works could then proceed in the open with full access for cranes and other equipment. Following completion of the internal structures to the proper point, the upper portion of the shell could be constructed to complete the containment vessel and then pressure tested in a manner comparable to modern construction practices instead of threading the entire plant through the eye of a needle.

Materials

In the choice of proper steels to be utilized in these designs, consideration must be given to minimizing the possibility of brittle failure of the steel. The use of steels which are known to be particularly susceptible to brittle failure should be eliminated. Most of the containment vessels designed to date have utilized SA-201 Grade B steels conforming to ASTM Specification A-300, which provides for heat treating to normalize the grain structure and for determining by Charpy tests the resistance to impact loadings at low temperatures of the material. In a recent interpretation of the ASME Unfired Pressure Vessel Code the use of materials conforming to this SA-300 Specification has become mandatory for containment vessels.

The importance of this protection against embrittlement cannot be overemphasized. Because of the fact that the Charpy tests which are commonly used to measure resistance to embrittlement depend upon the use of an impact loading condition, it is easy to fall into the erroneous assumption that brittle failures occur only under a shock loading. Contrary to this, it should be recognized that most pressure vessel brittle failures occur under static loadings

PO 6

rather than under impact loadings. The susceptibility to brittle failure is increased by the presence of discontinuities of the surface, points of restraint such as the bottom of a vessel resisting the expansion or contraction of the sides, overlaps of weld metals, notch conditions, or other conditions tending to establish a concentration of stresses. These are problems which are probably more severe in containment vessels than in many other types of pressure vessels because of the complications added by the remainder of the structure in a nuclear plant.

Air Locks and Penetrations

Proper provision must be made to allow entry of personnel, materials, piping and wiring into the shell without sacrificing its integrity. Personnel and material access is generally provided through an air lock, with two sets of doors which can be made airtight when closed. Interlocks are provided between the doors so that one cannot be opened unless the other is closed. This is accomplished by connecting shafts actuated by cams tied into the door locking mechanism or else by flexible push pull cables.

Air lock doors for containment vessels having a high-pressure differential are usually sealed by compressing a gasket with the door locking mechamism providing a clamping action. Air lock doors for low-pressure differentials have been designed with inflatable seals. Air locks for high-pressure installations become quite expensive with the cost running up to $30,000 to $50,000, and it becomes impractical to construct large locks for the access of major pieces of equipment. Large material handling openings are generally provided by a single bolted and gasketed door which cannot be opened during the time that the plant is operating.

Piping penetrations offer no great problem, since these can be welded to the shell unless temperature conditions prevent it. In that case, they can pass through a sleeve in the shell and be sealed to the sleeve with an expansion joint. Care should be taken to insure that the piping does not exert any appreciable thrust against the shell from expansion. Making proper provision for sealing penetrations of wiring is a difficult problem. Leakage may occur between the cable and the vessel wall, in the spaces between strands of stranded conductors and through the space between the cable and its insulation. Early plants have utilized individual conduits through the shell with pressure type fittings for each individual cable. On a large plant, where several hundred cable may pass through the shell, such an arrangement becomes quite complicated. As an alternative, an arrangement has been used whereby cables are grouped and passed through the shell in a cable pan, as shown in Fig. 4. The cable pan is then filled with a compound which hardens and adheres to the cable and walls of the pan. This has been successful, but depends upon proper laying of the cables to obtain complete flow of the sealing compound around each cable, and careless workmanship can cause major difficulties. Another system developed places an airtight box against the shell and allows the cables to pass through this box, with pressure fittings in the two walls through which the cables pass. This groups the cables somewhat, eliminates the dependency upon a sealing compound and affords an opportunity for a simple test on the integrity of the seals, since the box can be pressurized and checked for leaks.

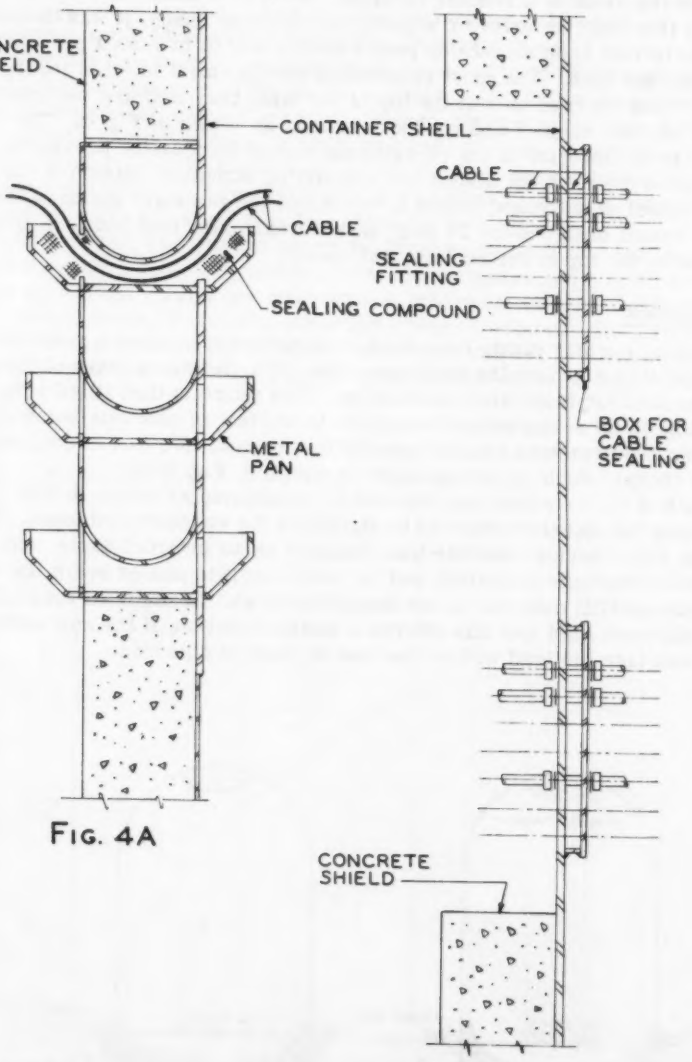


FIG. 4A

FIG. 4B

FIG. 4 CABLE SEALS

Water Storage

Many arrangements provide for spraying large quantities of water into the plant in the event of a reactor incident. Several of these have relied on obtaining this from an exterior source, but, in some cases, it was thought undesirable to rely upon an outside power source and to furnish a permanent overhead storage tank. The most economical arrangement for this incorporates an overhead storage tank at the top of the tank, thus utilizing the containment vessel as both support and roof for the tank as shown in Fig. 5. The tank is vented to the interior of the containment vessel to equalize pressures and thus eliminates the need for a separate supporting structure outside of the containment vessel of sufficient height to overcome the pressure inside of the vessel. For a vessel designed for 25 psig, an external water tank must be at least 60 feet above the top of the containment vessel.

Foundations

Several nuclear plants have been constructed by placing a major foundation slab and then erecting the shell upon this, with clearance between the steel and the slab for inspection and testing. This space is then filled with concrete resulting in an arrangement having six to ten feet of concrete below the vessel. Interior requirements usually provide for three to five feet of concrete inside of the vessel. Such an arrangement is shown in Fig. 5 (b).

Much of the exterior concrete can be eliminated as shown in Fig. 5 (a) by designing the interior concrete to distribute the concentrated loads. In this design, this interior concrete was designed as an inverted dome, with some localized bending considered, and the only concrete placed below the shell was unreinforced fill concrete in the inspection area. Loads from these plants are relatively light and this affords a simple solution, if the soil under the plant can take the load without the use of piles or caissons.

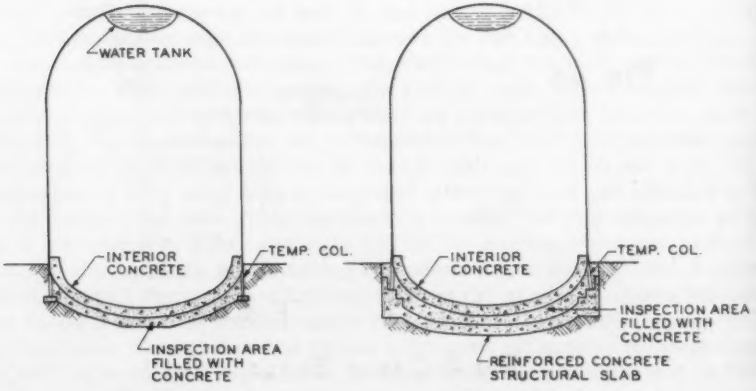


FIG. 5 A FIG. 5 B

FIG. 5 FOUNDATION STUDY

Economies of Plant Layout

The layout of the plant must be considered carefully in determing size and shape of the containment vessel. This should be studied carefully for each individual installation, as many factors must be considered in addition to the ones discussed in the foregoing paragraphs. Different types of plants will require varying arrangements of equipment and floor area requirements will differ at various elevations. This can be a major factor in the selection of the shape of the vessel, since a spherical vessel offers a large area for floors near the equator, but gives considerably less area at lower elevations, while an upright cylinder is more uniform in areas available at different elevations. Thus, where operating floor requirements are similar at various elevations, the use of a cylindrical vessel may be preferable to a sphere.

On the other hand, recognition should be given to the fact that the shape of the vessel does have an effect upon the total steel requirements and resultant cost. For a constant volume and pressure, a sphere utilizes less plate than any other shape and is, therefore, generally cheaper, although the higher cost of dished plates in place of rolled plates and added support problems tend to offset this to some degree. Fig. 6 gives an indication of the savings in weig

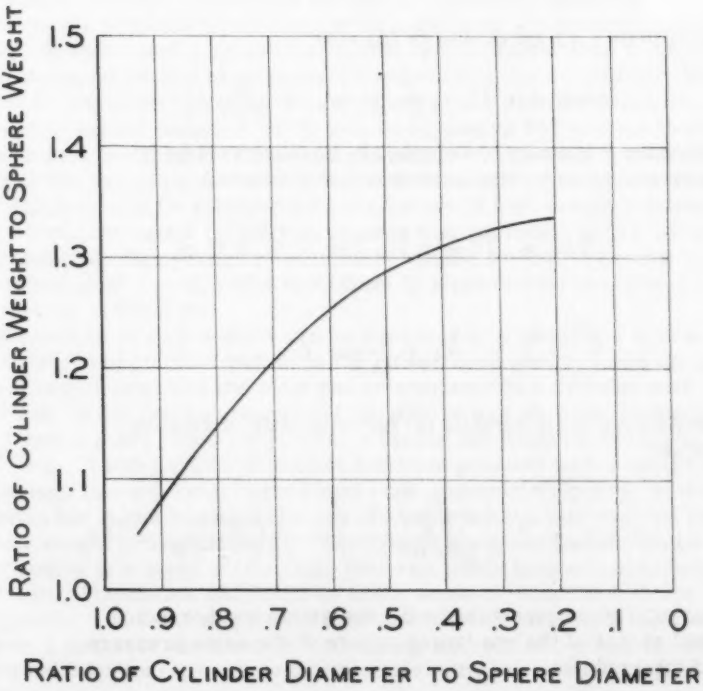


FIG. 6 COMPARISON OF WEIGHT OF CYLINDER WITH HEMISPHERICAL HEAD TO SPHERE OF SAME VOLUME

made possible by a sphere. This figure compares the weight of a cylinder with hemispherical ends to a sphere of the same volume and shows the effect of reducing the diameter, but increasing the length to maintain the same volume. A similar relationship exists for a cylinder with one hemispherical head and one ellipsoidal or torispherical head, except that the ratio of weights would be greater because of the increased thickness of an ellipsoidal or torispherical head.

An interesting point is the fact that if pressure, shape of vessel and volume are maintained constant, it makes no difference in total weight if one vessel or several smaller ones of the same shape are used. This is illustrated in the case of spherical vessels by considering the formula for thickness of plate in a spherical shell:

(1)
$$t = \frac{PR}{2SE - 0.2P}$$

where t = thickness
P = Pressure psig
R = Radius
S = allowable stress
E = joint efficiency

total plate = t x surface area (A) = At

(2)
$$At = 4\pi R^2 \frac{PR}{2SE - 0.2P} = \frac{4\pi R^3 P}{2SE - 0.2P}$$

Consider 2 spheres of Volume $2V_1$ equal to Volume V of larger sphere. Pressure P remains constant.

(3)
$$2V_1 = 2\left(\frac{4}{3}\pi R_1^3\right) = V = \frac{4}{3}\pi R^3$$

(4)
$$R_1 = \frac{1}{\sqrt[3]{2}} R$$

Substituting (4) in formula (2) for total plate of smaller shells.

(5)
$$2A_1t_1 = 2\,\frac{4\pi\left(\frac{R}{\sqrt[3]{2}}\right)^3 P}{2SE - 0.2P} = \frac{4\pi R^3 P}{2SE - 0.2P} = At$$

thus total plate quantities in the two smaller spheres are equal to that of the one larger sphere of the same pressure and total volume.

The situation is considerably different, however, when a change in design pressure is considered. From equation (2) above it can be seen that the total plate quantity for a sphere varies directly with the pressure and with the cube of the radius. Thus, if the pressure varied inversely, as the cube of the radi

(inversely as the volume), the total plate quantity would be unchanged no matter what size sphere was selected. This would mean then that the gas would have to follow the following law:

$$P_1 V_1 = P_2 V_2$$

This, however, is not the case as shown in Fig. 1. Instead there is a different relationship entirely as the amount of water vapor released to the mixture of air and water vapor varies with the pressure so that actually the quantity of gas is varying as well as the pressure and volume. The steep slope of the curve at low-pressures is due to the presence of an initial quantity of air which changes the slope at lower pressures, but has little effect at higher pressures. If the curve for one of the initial conditions of the hot water in the system is considered and values for the pressures at various points are selected, and substituted in equation (2), the results show that the total plate quantities vary. Fig. 7 shows a curve for quantity of plate in a spherical vessel per pound of pressurized water in the system for various conditions of final pressure in the containment vessel. A curve is also shown for a cylindrical vessel, with hemispherical ends, of radius R_1 equal to 80% of the radius for a sphere of equal volume and pressure.

This curve presents some interesting aspects on the economics of different shapes. It shows that a cylindrical vessel is 15% heavier than a spherical vessel designed for the same pressure and volume, but is actually lighter than spherical vessels designed for larger volumes and smaller pressures. Thus, a spherical vessel designed for 30 psia or 15 psig is 55% heavier than a cylindrical vessel designed for 60 psia or 45 psig to hold the same quantity of gas.

From the preceding discussion it appears that major economies in the cost of containment can be achieved through the use of high design pressures. If this is done, the use of cylindrical vessels may, at times, prove advantageous, since this will often permit reductions in volume without impairing the efficiency of the plant layout, while reductions in sizes of spheres often cramp the lower areas of the plant.

An example of such a study can be seen in Fig. 8 showing a typical plant with a sphere of 100 foot radius as the containment shell. From the arrangement it can be seen that the plant can be contained in a cylinder with a radius of 76 feet. If the design pressure of the sphere was 15 psig, we can see from Fig. 1 that the 30% reduction in volume causes the design pressure to increase to 20 psig. If this savings in volume had been possible with a smaller sphere, the savings in steel weight would have been approximately 20%. In this case, however, the layout requires the use of a cylinder and this reduces the savings in steel weight to approximately 10%. There are other factors which should be evaluated in a study of this type, however, since they will also bear on the final cost. Foundation and concrete costs would be reduced with the use of the cylinder and these might be a determing factor. If conditions are such to require a concrete shadow shield lining the interior of the vessel. The savings with a cylinder increase since this smaller surface area will require less concrete.

If the initial design pressures for this study had been in the neighborhood of 30 psig, the design pressure for the cylinder would be 42 psig and the cylinder would have been heavier. From Fig. 7 it can be seen that as the design

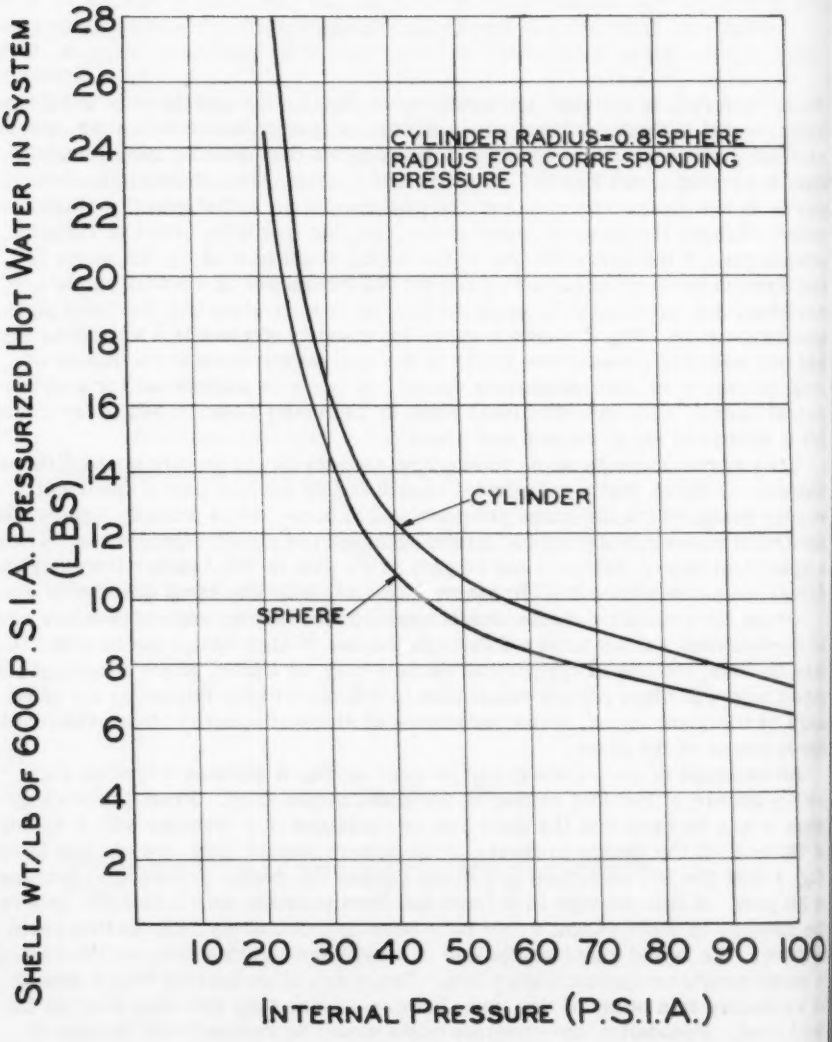


FIG. 7 CONTAINMENT SHELL PLATE QUANTITIES PER POUND OF WATER IN SYSTEM

pressure reaches the range of about 50 psig, the curve flattens and the steel savings of the sphere over the cylinder of almost any diameter become considerable.

It can be seen, from the foregoing, that each installation must be studied carefully in order to obtain the most economical arragnement. There is evidence, however, that the use of higher design pressures will result in the use of less steel. Whether or not this will result in a net savings will depend upon many factors, including requirements for field stress relieving vessels of thick walls.

REFERENCE

1. "Review of Present American Design of Containment Vessels for Nuclear Power Plants" by Alf Kolflat. Harrogate, England, June, 1958.

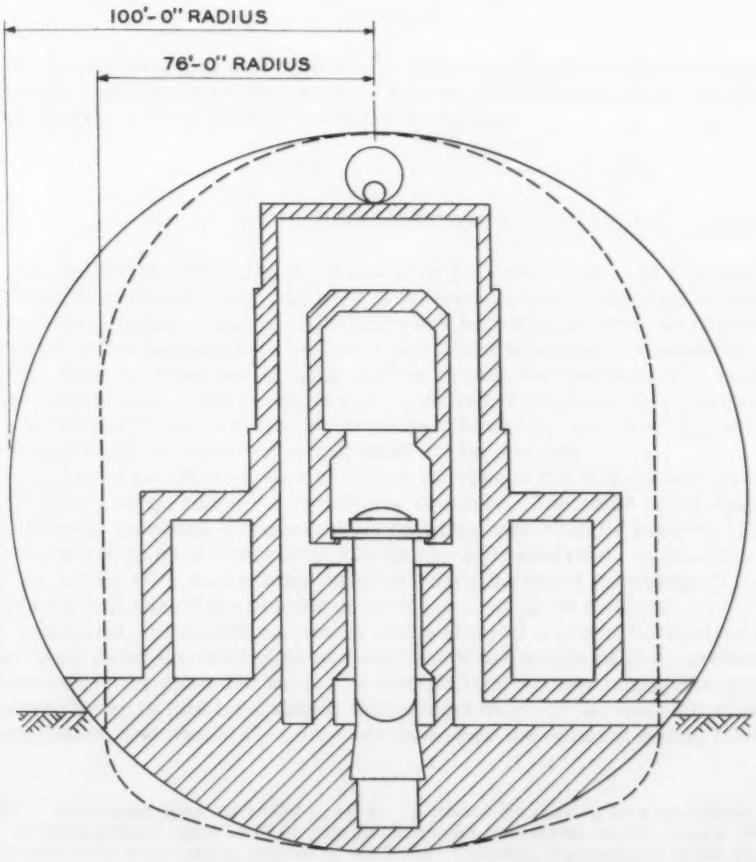


FIG. 8 STUDY OF SHELL LAYOUT

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

DETERMINATION OF STRESSES ON ANCHOR BLOCKS

M. R. Bouchayer [1]

SYNOPSIS

By the use of mathematical formula, given in this paper, for the design of penstock anchorages and by following a more accurate determination of operating forces, a more economic system is obtained.

INTRODUCTION

The determination of the thicknesses of the various parts of a pressure pipeline is a subject which has been discussed on many occasions in technical literature. Certain types of calculation are becoming of ever-increasing interest to the designers as well as to the manufacturers or consulting engineers, and due to the use of mathematical processes now available, considerable progress has been made, and has resulted in much better utilization of the metal employed on these constructions; however, the "poor parent" of the pressure pipeline, concrete itself, must not be ignored.

To ensure the stability of a pressure pipeline it has always been necessary to employ a large volume of reinforced concrete. The notes which follow are designed to show that considerable economics may often be realized if the stresses set up on the anchoring blocks are fully analyzed. A secondary aim of the article is to suggest that in other cases the exact knowledge of these stresses will enable the amount of steel required to be reduced.

In general, it appears necessary that a study of a pressure pipeline should not come under the head of an examination of the weight of steel necessary, as is generally the case, but should be viewed from a wider angle; the specialized manufacturer in this field should concentrate more on the over-all view of the installation, and should thus offer his customer the solution which, taken by

Note: Discussion open until May 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2291 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 6, December, 1959.

1. Director General, Societe Dauphinoise d'Etudes Et DeMontages, Paris, France.

and large, and including the civil engineering aspect, is found to be the most economical from all points of view.

The Action of the Forces Previously Mentioned on the Pipeline and on its Fixed Points

The external forces operating on any type of pipeline may be considered from two aspects:

Either as:

- Permanent forces
- Intermittent forces
- Exceptional forces

Or as:

- Static forces
- Dynamic forces

The three types of forces first mentioned bear on the factor of safety which is taken into account in the calculations, while the two latter categories cause different effects, and here the calculation methods are not the same.

Permanent Forces

The permanent forces normally come into operation when the pipeline is in service at its calculated pressure. These forces are:

- the weight of the filled pipeline and possibly the additional weight of the earth for buried pipelines,
- the hydraulic forces created by the water on:
  - elbows,
  - bottom sections and cutoff devices,
  - joints,
  - converging or diverging sections,
  - thermal stresses due to the variations of the temperature of the water,
  - frictional forces resulting from displacements set up by the other forces.

Intermittent Forces

The intermittent forces operate temporarily, under conditions which occur relatively infrequently. These forces are:

- thermal stresses operating on an empty pipeline,
- hydraulic forces due to the setting up of testing pressures on site,
- forces due to an external pressure, in the case of a pipeline in a tunnel.

Exceptional Forces

Exceptional forces operate under abnormal and rarely encountered conditions. These forces are:

- hydraulic forces due to the setting up of test pressures in the works,
- forces due to atmospheric pressure in the case of a pipeline subjected to absolute vacuum conditions,
- earthquake forces,
- All other exceptional forces, which may be set out in the specification; for example, the subsidence of a support in the case of a pipeline on spaced supports, or the failure of the rock or concrete supports in the case of a pipeline in a tunnel, etc.

Static Forces

The action of static forces is set up slowly. In this connection we have to consider forces due to weight, the overburden of the earth, to thermal variations, and to static pressure of the water. In a normal service these forces are constant, or at any rate it is more or less certain that between them they will vary slowly and within small limits.

Dynamic Forces

The action of dynamic forces is set up in a very short period, and their intensities during this period may vary within wide limits. In this connection we have to consider water hammer, earthquake forces, and the action of atmospheric pressure on an empty pipeline.

All the external forces previously mentioned operate on the pipeline, and through the medium of the pipe, have repercussions on the anchorages and on the supports. These supports are, in addition, subjected to two other forces, their own weight, and the reactions of the ground on which they are constructed. It is not, in general, possible to separate the study of the pipeline from the study of its anchorages and supports, since the position or the spacing of the latter has a considerable influence on the stresses set up in the pipe, and also conditions their reaction on the pipe on its fixed points.

The pipeline is thus a complete assembly, and the calculation of the pipe itself should be completely integrated with that of its anchorages. This interaction between the two thus leads the designer, after having established and dimensioned his anchorages, to check their influence on the behavior of the steel of the pipeline.

Mode of Action of the Forces
Choice of the Stresses

For the steel of the pipeline, the specifications will in general call for different coefficients of safety according to whether the forces act in a permanent fashion or whether they are intermittent or exceptional. The choice is made in each of these conditions, on the basis of the most unfavorable circumstances, and the working stress of the most highly burdened fibre is calculated.

For the anchorages, the most unfavorable combination of forces is similarly considered, and in general the designer is led to check the anchorage under many different conditions of service, each condition being studied under the most unfavorable circumstances.

Consideration must be given to all cases, according to the type of pipeline (welded, hooped, etc.) the conditions of filling, and the conditions of installation.

1°)-According to the type of pipeline

A. Welded pipeline with expansion joints.
In this case the elbows are anchored (Swiss method); the following table shows the various forces set up in this case.

Summit 1

1. Component of the inherent weight of the downstream part of the pipeline up to the expansion joint.

2. Frictional forces between the conduit and the supports of thermal origin, or by longitudinal contraction under internal pressure.

3. Frictional forces at the expansion joint, of thermal origin, or by longitudinal contraction under internal pressure.

4. Hydrostatic pressure on the male ring of the expansion joint.

Elbow Number 2

1. Hydrostatic pressure on the internal section of the upper pipe.

2. Hydrostatic pressure on the internal section of the lower pipe.

3. Hydrostatic pressure on the female ring of the upstream expansion joint.

4. Hydrostatic pressure on the male ring of the downstream expansion joint.

5. Frictional forces on the upstream expansion joint, of thermal origin or by longitudinal contraction under internal pressure.

6. Frictional forces on the downstream expansion joint, of thermal origin or by longitudinal contraction under internal pressure.

Elbow Number 3

1. Hydrostatic pressure on the internal section of the upper pipe.

2. Hydrostatic pressure on the internal section of the lower pipe.

3. Hydrostatic pressure on the female ring of the upstream expansion joint.

4. Hydrostatic pressure on the male ring of the downstream expansion joint.

5. Hydrostatic pressure on a possible upstream reduction cone.

6. Component of inherent weight of the upstream pipe as far as the expansion joint.

7. Component of inherent weight of the downstream pipe as far as the expansion joint.

8. Frictional forces on the pipeline supports of thermal origin or by longitudinal contraction under internal pressure, in the upstream pipeline.

9. Frictional forces at the upstream expansion joint, of thermal origin or by longitudinal contraction under internal pressure.

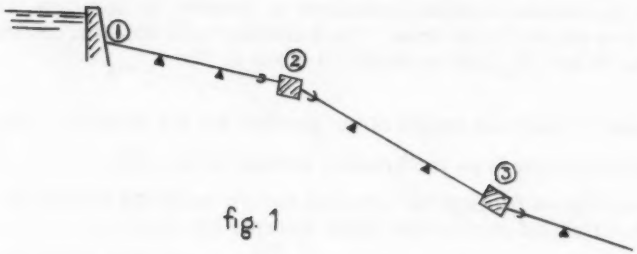


fig. 1

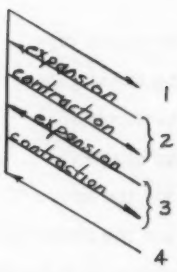


fig. 2

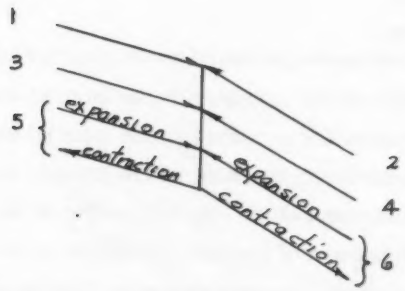


fig. 3

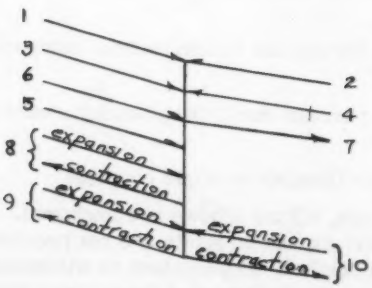


fig. 4

10. Frictional forces at the downstream expansion joint, of thermal origin or by longitudinal contraction under internal pressure.

B. Welded pipeline or overpressured (without expansion joints)
This is the method currently employed in France. In this case, the elbows are allowed to be free. The following table considers a free elbow at (2) and (4), and an anchored elbow at (3):

Summit (1)

1. Component of inherent weight of the pipeline for the zones (1) - (2).

2. Hydrostatic pressure on the internal section of (1) - (2).

3. Frictional forces between the pipeline and the supports of thermal origin or by longitudinal contraction under internal pressure.

4. Thermal forces or forces due to longitudinal contraction under thermal pressure.

Elbow (3)

1. Hydrostatic pressure on the internal section of the upper pipe.

2. Hydrostatic pressure on the internal section of the lower pipe.

3. Hydrostatic pressure on the internal section of the elbow (2).

4. Hydrostatic pressure on the internal section of the elbow (4).

5. Component of the inherent weight of the pipeline for the zones (2) - (3).

6. Component of the inherent weight of the pipeline for the zones (3) - (4).

7. Frictional forces between the pipeline and its supports of thermal origin or longitudinal contraction under internal pressure for the upstream portion.

8. Frictional forces between the conduit and its support of thermal origin or by longitudinal contraction under internal pressure for the downstream portion.

9. Thermal forces or forces due to longitudinal contraction for the upstream portion.

10. Thermal forces or forces due to longitudinal contraction for the downstream portion.

C. Self-hooped pipeline (flexible or rigid hooping)

In the case of this design, all the elbows are anchored. Unless specially requested by the customer, expansion joints are not provided. The anchorages at the elbows enable the self-hooped pipes to withstand the longitudinal forces due to the pressure, the thickness of these pipes being sufficient to absorb these forces.

Summit (1)

1. Thermal forces or forces due to longitudinal contraction under internal pressure.

2. Component of the inherent weight of the pipeline.

Elbow (2)

1. Hydrostatic pressure on the internal surface of the upper pipe.

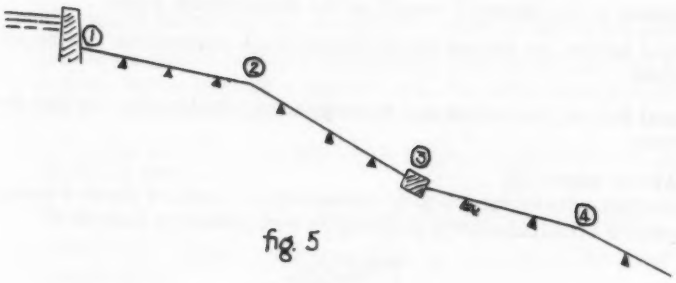


fig. 5

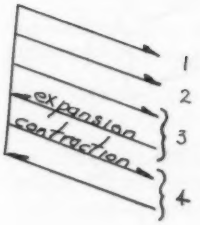


fig. 6

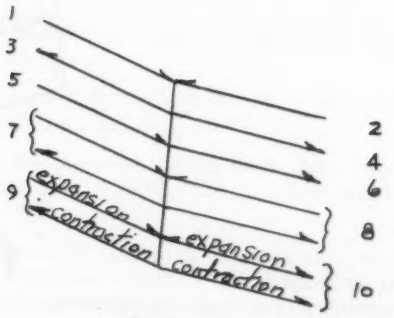


fig. 7

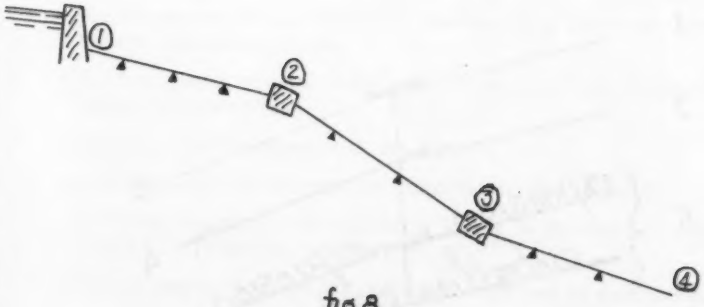


fig. 8

2. Hydrostatic pressure on the internal surface of the lower pipe.

3. Component of the inherent weight of the upstream pipe.

4. Component of the inherent weight of the downstream pipe.

5. Thermal forces, or forces due to longitudinal contraction for the upstream portion.

6. Thermal forces, or forces due to longitudinal contraction for the downstream portion.

Elbow (3) As for elbow (2)

Before analyzing these forces it is necessary to consider those amongst them that operate simultaneously, and this is considered in section 2°.

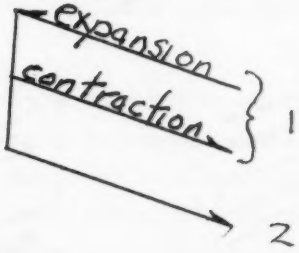
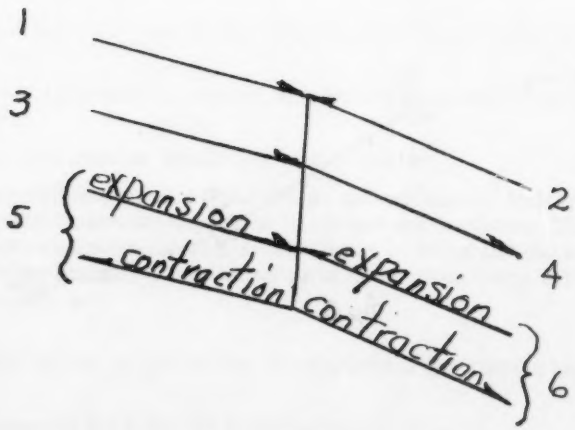


fig. 9



2°)-According to the conditions of filling of the pipe, and of installation.

A. Pipeline empty -
The following forces must be taken into account:

- the component along the axis of the weight of the empty pipeline,
- frictional forces between the pipeline and the supports of thermal origin, (in the case of a pipeline equipped with expansion joints; the forces are negligible in other cases),
- frictional forces at the expansion joints, in the case of a pipeline provided with joints,
- thermal forces except in the case of pipelines with joints.

B. Pipeline full, without pressure (that is, filled up to the point under consideration)
The following forces must be taken into account:

- the component along the axis of the weight of the pipeline for the upstream and downstream portions,
- the component normal to the axis of the weight of water contained in the pipeline for the downstream portion,
- the frictional forces between the pipeline and the supports, of thermal origin, for the upstream empty part, which is the only part subjected to thermal stresses,
- the frictional forces at the expansion joints, in the case of a pipeline provided with joints (for the upstream part; forces of thermal origin, for the downstream part; thermal forces plus contraction forces)
- thermal forces for the upstream portion and for the downstream portion, except for pipelines provided with joints.

C. Pipeline full, under pressure.

- the component along the axis of the weight of the empty pipeline upstream and downstream,
- the component normal to the axis on the weight of water in the section under consideration,
- the hydrostatic pressure at the elbows,
- frictional forces between the pipeline and the supports,
- frictional forces at the expansion joints, in the case of a pipeline provided with joints (due to contraction),
- forces due to longitudinal contraction caused by internal pressure,
- hydrostatic pressure on the expansion joints,
- centrifugal force of the water at the elbows,
- thermal forces upstream and downstream.

For each case A, B, C, above, the resultants of these forces are worked out.

3°)-According to the emplacement of the pipelines.

A. Pipelines above the ground.
All that has been set out above applies in this case.

B. Buried pipelines.

It is wise to consider the pipeline not being covered before it is placed into service. If this arrangement is made, the forces acting on it are the same as for a conduit above the ground.

The filling covering the pipeline does not have any influence on the stability of the anchoring blocks, and does not transmit any axial force to the pipeline.

C. Concreted pipelines.

The same points apply here as in "B," providing the covering with stonework does not take place until after the pipeline has been placed in service. The coating does not transmit any axial force to the pipeline, and it may possibly oppose, if the adherence between the coating and the pipeline is satisfactory, the transmission of forces due to longitudinal contraction through internal pressure; but this action only operates in the direction of improving the safety conditions.

D. Pipelines in a tunnel (resting on blocks).

Thermal forces do not come into play in this case, except towards the lower entrance to the tunnel, where the existence of a rising current of air (if the tunnel is not closed) sets up a zone of variable temperatures both in summer and in winter.

Calculation of the Various Forces

A. Weight of the filled pipeline

1. Pipeline with joints, anchored at the elbows.

The axial component of the weight of the water is transformed into pressure. The only axial component then operating will be

$P_v \sin \alpha$,

P_v being the weight of the empty pipe,

The component normal to the axes will be $P_p \cos \alpha$,

P_p being the weight of the filled pipe.

2. Pipeline without joints, anchored at the elbows.

The weight is applied at G, the center of gravity of the section of pipeline between the two anchorages.

If the section varies considerably between the two anchorages, the mathematically correct expression is as follows:

- Force on the downstream block: $$F_1 = P_v \sin \alpha \frac{S_1}{l_1 \left(\frac{S_1}{l_1} + \frac{S_2}{l_2}\right)}$$

- Force on the upstream block will be:

$$F_2 = P_v \sin \alpha \frac{S_2}{l_2 \left(\frac{S_1}{l_1} + \frac{S_2}{l_2}\right)}$$

with $S_1 = \dfrac{l_1}{\sum \frac{l_i}{S_i}}$ for the portion GB

and $S_2 = \dfrac{l_2}{\sum \frac{l_i}{S_i}}$ for the portion GA

S_1, S_2 are the equivalent sections of the metal for the spans GB and GA

If the section varies very slightly between A and B, the following will apply:

$$F_1 = \frac{P_v \sin \alpha}{2}$$

$$F_2 = \frac{P_v \sin \alpha}{2}$$

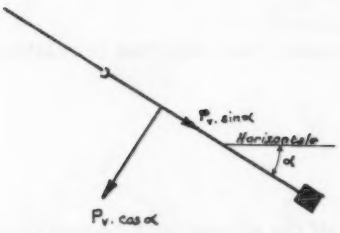


fig.11

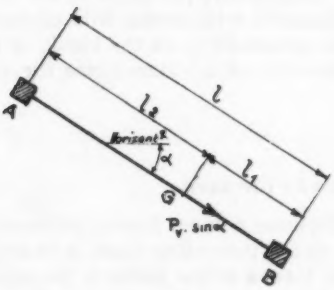


fig.12

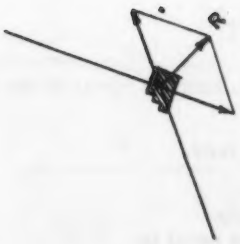


fig.13

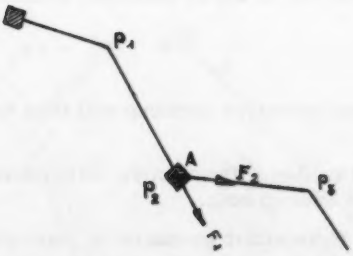


fig.14

3. Pipeline without joints, with free elbows.
See special note in connection with these pipelines.

4. Frictional forces
The component $P_p \cos \alpha$, normal to the pipeline, will create a frictional force which will curtail the total force.

B. Weight of the empty pipeline

The axial forces are the same as in the preceding case. On the other hand, the empty pipeline, with free elbows, may buckle slightly towards the top under the influence of the thermal forces so that the frictional force $c \times P_v \cos \alpha$ does not operate integrally; it is thus prudent not to subtract this frictional force from the total force.

C. Hydraulic forces

1. Hydrostatic pressure at the elbows
- Pipeline with joints, with elbows anchored
The pressure R, on the block, is the geometrical resultant in relation to the bisection of the axes of the forces:

$$p \times \frac{\pi D^2}{4}$$

carried by the axes.

- Pipeline without joints, anchored at all the elbows (self-hooped pipelines).
As in the preceding case, it is desirable to avoid all transmission of longitudinal forces to the metal of the pipeline. In the preceding case, the joint could not transmit these forces, but in the case of a self-hooped pipe, the fatigue of the metal will be dangerously increased. The pressure R is obtained as in the preceding case.

- Pipeline without joints, anchored at certain elbows (welded pipeline).
At A, the pressure due to the upstream branch will be:

$$F_1 = (p_2 - p_1) \frac{\pi D^2}{4}$$

That due to the downstream branch will be:

$$F_2 = (p_3 - p_2) \frac{\pi D^2}{4}$$

Supplementary tensions will thus be introduced into the metal of the pipeline.

- Pipeline without joints, with elbows entirely free.
See special note.

2. Hydrostatic pressure on free bottom sections.
This pressure is directed along the axes and is equal to:

$$p \frac{\pi D^2}{4}$$

3. Hydrostatic pressure on convergent or divergent sections.

If D_1 and D_2 are the two diameters of the pipeline, the hydrostatic force on the convergent or the divergent section is directed along the axes and is equal to:

$$p \frac{\pi}{4} (D_1^2 - D_2^2)$$

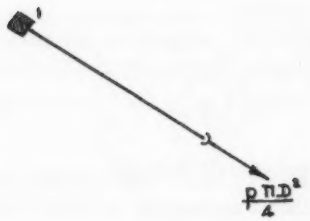
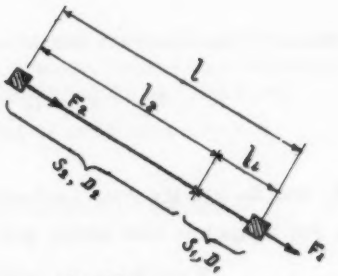


fig.15



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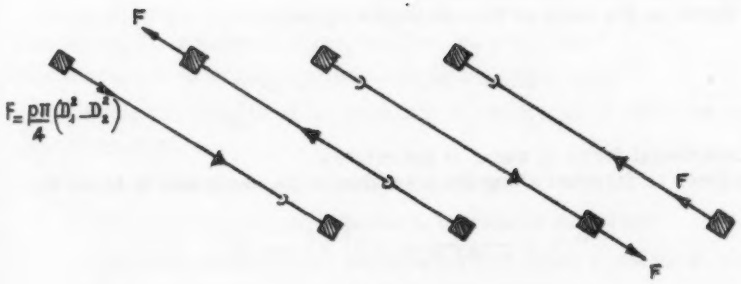


fig. 17

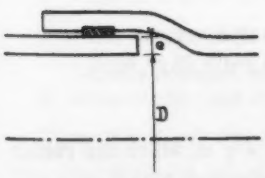
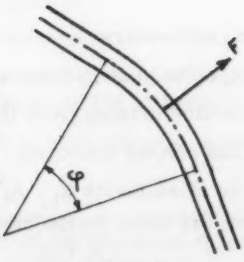


fig.18



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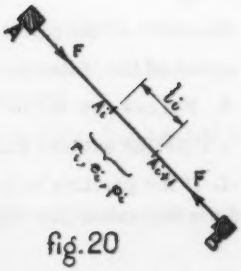


fig.20

directed towards the downstream or the upstream side, according to whether it is a question of a convergent or a divergent section.

- For a pipeline without joints, and with elbows anchored.

Force on the downstream block:

$$F_1 = p \frac{\pi}{4} (D_1^2 - D_2^2) \frac{S_1}{l_1 (\frac{S_1}{l_1} + \frac{S_2}{l_2})}$$

Force on the upstream block:

$$F_2 = p \frac{\pi}{4} (D_1^2 - D_2^2) \frac{S_2}{l_2 (\frac{S_1}{l_1} + \frac{S_2}{l_2})}$$

S_1 and S_2 are the equivalent sections previously defined.

- For a pipeline with joints, and with elbows anchored:

The force $p \frac{\pi}{4} (D_1^2 - D_2^2)$ will be transmitted only to one of the blocks according to its position.

4. Hydrostatic pressure on the joints:
The force on the male or female joint ring is:

$$F = \frac{p\pi}{4} \left| \begin{matrix} \\ \\ \\ \end{matrix} \right. (D + 2e)^2 - D^2 \left| \begin{matrix} \\ \\ \\ \end{matrix} \right.$$

5. Centrifugal force of water at the elbows.
This force is directed along the bisection of the angle and is equal to:

$$F = \frac{\pi}{2 \times 981 \times 10^3} D^2 V^2 \sin \frac{\varphi}{2}$$

$$F = \gamma \frac{\pi D^2 V^2}{2g} \sin \frac{\varphi}{2}$$

F in kg

D diameter of the pipeline in centimeters

V speed of the water in the pipeline in centimeters per second

6. Forces due to the contraction arising from the Poisson effect.

- Pipeline without joints, and elbows anchored.

1. If the pipeline is made up of elements $A_i \cdot A_{i+1}$ in which the radius and the thickness are constant, the force on the blocks will be:

$$F = 1{,}88 \frac{\Sigma \frac{r_i l_i p_i}{e_i}}{\Sigma \frac{l_i}{r_i e_i}}$$

p_i is the pressure at the center of the element under consideration.

2. If the pipeline has a radius and a thickness which are both sensibly constant between the two blocks:

$F = 0.471\ p_m D^2$, p_m being the pressure in the center of AB

- Pipeline without joints, with elbows free.
See special note.

D. Thermal Forces

- Pipeline without joints, with elbows anchored.

A contraction or dilation due to thermal reasons, caused by a decrease or an increase of temperature $\pm\ \Delta D$, causes on the blocks an axial force

$$F = 0.232\ \Delta D\ \frac{1}{\sum \frac{l_i}{S_i}}$$, S_i being the section of metal of a length l_i.

If the pipeline has a steel section S which is nearly constant,
$F = 0.232\ \Delta D \times S$

- Pipeline without joints, with elbows free.
See special note.

E. Frictional Forces

1. Friction of the pipeline on its supports.
This force, for a filled pipeline, will be: $p_p \cos \alpha \tan \rho$
This force, for an empty pipeline, will be : $p_v \cos \alpha \tan \rho$

p_p, p_v being the weights of the pipe full or empty, $\tan \rho$ being the coefficient of friction.

$\tan \rho$ = 0.1 for supports on lubricated rollers

= 0.1 to 0.2 for supports between lubricated surfaces

= 0.3 to 0.5 for supports between nonlubricated metallic surfaces

= 0.5 to 0.7 for supports between rusty metallic surfaces

2. Friction between the packing of the expansion joints.
Let:

l = the length of the packing

D = the diameter of the pipe

p = the minimum pressure of the packing to ensure watertightness (this is at least equal to the utilization pressure).

K = the coefficient of friction between steel and the packing = 0.3 to 0.5

Then $F = K \pi D l p$

The total force on the block will be attained by subtracting from the sum of the axial forces, the sum of the frictional forces.

Forces Taken Into Consideration Under Practical Conditions

In general, the preponderating forces are as follows:

The hydrostatic pressures on elbows, bottom sections, convergent and divergent pieces,

The longitudinal contraction due to the Poisson effect,

The thermal forces.

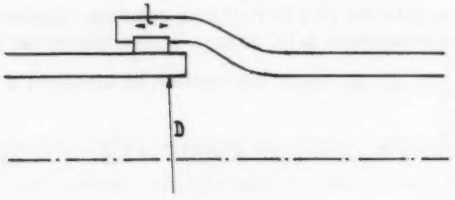


fig. 21

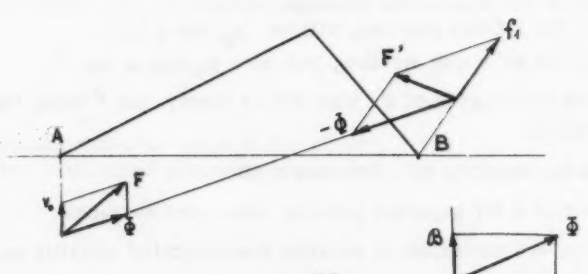


fig. 22

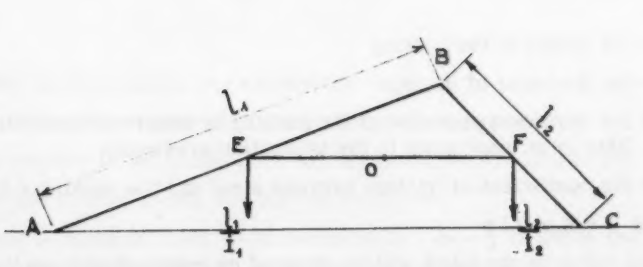


fig. 23

It is wise to take account of the size of other forces in order to judge whether or not they may be totally neglected.

The study of the behavior of the blocks is then undertaken with regard to the following cases:

1 - Pipeline full under test pressure with the water immobile,

2 - Pipeline full under utilization pressure, (including water hammer) with the water in movement,

3 - Pipeline in the course of being filled,

4 - Pipeline empty.

Study of the Forces Acting on a Pipeline with Free Elbows

The study of a pipeline with free elbows cannot be carried out in the same manner as that for a pipeline with anchored elbows, for which every part of the pipeline between two anchorages is rectilinear.

When the condition of free elbows between anchorages is introduced, the situation resembles that of an arc, in the general sense, that is to say of a beam along the line of the mean curve. This means the introduction of moments of flexion, and eccentric forces, and it is also necessary to take into consideration the elasticity of the elbows.

General Method of Calculation

The arc thus taken into consideration is hyperstatic, and it is fixed at its two ends, which are the anchoring blocks. The method of calculation consists of working out the reactions of the arc when rendered isostatic, and to add to these reactions a complementary reaction made up of the horizontal component Q and the vertical component β together with the moment $\mathcal{A}$ in relation to a point taken as the origin of the coordinates.

The reference arc is an arc made isostatic by the replacement of A by a simple and of B by a knee-joint.

The reactions of the isostatic arc are v_0 and f_1, and the complementary reactions are Φ and $-\Phi$. It can be shown that they are acting in the same absolute value, but of opposite sign. The reactions of the hyperstatic arc are thus the resultant F and F', in magnitude and in position.

The values of $\mathcal{A}$, β and ρ are given by:

$$\mathcal{A} = -\frac{\int \frac{\mu\, ds}{i}}{\int \frac{ds}{i}} \quad , \qquad \beta = -\frac{\int \frac{\mu\, x\, ds}{i}}{\int \frac{x^2\, ds}{i}} \quad , \qquad \rho = \frac{E\, c\, \Delta D_1 + \int \frac{\mu\, y\, ds}{i}}{\int \frac{y^2\, ds}{i} + \int \left(\frac{dx}{ds}\right)^2 \frac{ds}{\Omega}}$$

In the equations μ represents the moment at a point of the isostatic arc, c is the coefficient of expansion of the steel ΔD is the difference of temperature at the origin, 1 the span of the arc, i the moment of inertia of a section, and Ω is the surface of a section. As μ is known together with F and F' in magnitude and position, the moment at each point can be deduced.

Simplifying Hypotheses Relative to the Structure of the Arc

Choice of the origin

In the preceding formulae, which give the values of α, β and ρ, the origin O is the center of gravity of the mean line of the hypothetical masses $\frac{ds}{i}$. This can easily be found by the usual methods of static graphical analysis, that is to say by the use of two dynamic and funicular polygons.

When the pipeline has a diameter which is constant between two anchorages, the variation of the thickness, and thus of the inertia, is sufficiently small to enable one to consider the hypothetical mass of each section concentrated at its center, that is $\frac{1_i}{I_i}$ at E, $\frac{1_2}{i_i}$ at F.

i_i and i_2 will then represent the equivalent inertia figures of each section, so that

$$i_1 = \frac{1_1}{\Sigma \frac{1_i}{i_i}}, \text{ from A to B, and } i_2 = \frac{1_2}{\Sigma \frac{1_i}{i_i}} \text{ from B to C}$$

For a pipe, i is approximately equal to π e r^3. The position of the center of gravity will not be changed, if the diameter remains constant, while finding the center of gravity of two hypothetical masses $\frac{1_1}{e_1}$, $\frac{1_2}{e_2}$, applied at E and F, e_1 and e_2 being the equivalent thicknesses such that from A to B

$$e_1 = \frac{1_1}{\Sigma \frac{1_i}{e_i}} \text{ from B to C, } e_2 = \frac{1_2}{\Sigma \frac{1_i}{e_i}}$$

Choice of axes of the co-ordinates

In the preceding formula, the axes of the co-ordinates are theoretically, at O, two axes conjugate to the central ellipse of inertia, one of these axes being parallel to the line of the anchorages A - C.

If the axes are perfectly symmetrical, these axes will be perpendicular. In practice, they may always be taken as such, on condition that the sections AB and BC are not unduly unequal, and that the moments of inertia i vary only slightly from A to C. These conditions are frequently encountered in pressure pipelines.

Simplification of the expression for the pressure ρ:

If the arc is not too greatly surbased, one may at the first approximation neglect the term $\int (\frac{dx}{ds})^2 \frac{ds}{\Omega}$ and one may then write:

$$\varphi = \frac{E c \Delta D l + \int \frac{\mu y \, ds}{i}}{\int \frac{y^2 \, ds}{i}}$$

In carrying out these mathematical operations, the tendency is to move in the direction of safety since in this way one obtains a pressure somewhat greater than the actual pressure.

Other simplifications of $\mathcal{A}$, β, ρ:

In order to avoid the calculation of over-complicated integrals the following integrals

$$\int \frac{\mu \, d_s}{i} \, , \int \frac{\mu \, x \, d_s}{i} \, , \quad \int \frac{\mu \, y \, d_s}{i} \, , \int \frac{x^2 \, d_s}{i} \, , \text{ etc . . .}$$

may be obtained by figures obtained by dividing the line of the arc into intervals Δ_s which are equal, and in which i, μ, x, y are considered as constant. Then

$$\mathcal{A} = - \frac{\Sigma \frac{\mu}{i}}{\Sigma \frac{1}{i}} \, , \quad \beta = - \frac{\Sigma \frac{\mu x}{i}}{\Sigma \frac{x^2}{i}}$$

$$\varphi = \frac{E \, c \, \Delta \, D \, 1 + \Delta s \Sigma \frac{\mu \, y}{i}}{\Delta s \quad \Sigma \frac{y^2}{i}}$$

These formula will allow, if required, the problems in this field or calculation to be solved in a manner which is sufficiently accurate.

Action of the Weight of the Filled Pipeline

As the component of the weight of the water operating along the line of the pipeline is transformed into pressure, the weight of the pipeline is resolved into two parts: G_1 weight of the empty pipeline, and G_2 the weight of the filled pipeline. G_A and G_N are thus the axial and normal components of the weight thus resolved, giving a resultant R.

The resultant R is then resolved parallel to the line of supports and also perpendicular to this line.

In this way the final components G_h, G_p are obtained.

Action of the Hydrostatic Force at the Free Elbow

The hydrostatic force F is in the same way resolved into P perpendicular to the line of fixings, and H parallel to this line.

Action of the Variation of Temperature

The pressure due at a variation of temperature acts on a straight line parallel to the line of fixings and passes through the origin previously defined (the center of gravity of the mean line in relation to the hypothetical masses $\frac{d_s}{i}$). Its value is, as already shown,

$$\varphi = \frac{E \, c \, \Delta \, D \, 1}{\int \frac{y^2 \, d_s}{i}} \quad \text{or} \quad \frac{E \, c \, \Delta \, D \, 1}{\Delta s \, \Sigma \frac{y^2}{i}} \quad \text{as an approximate value}$$

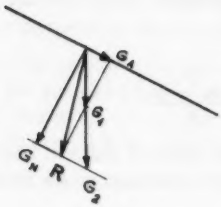


fig. 24

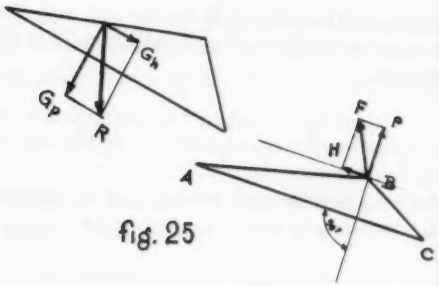


fig. 25

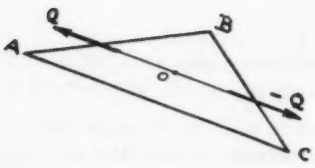


fig. 26

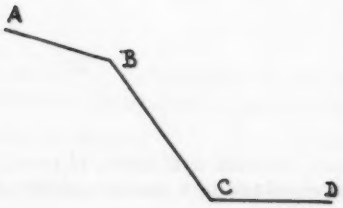


fig. 27

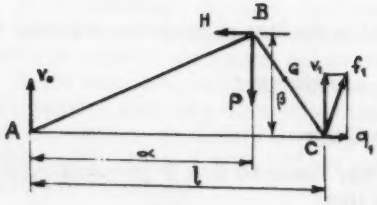


fig. 28

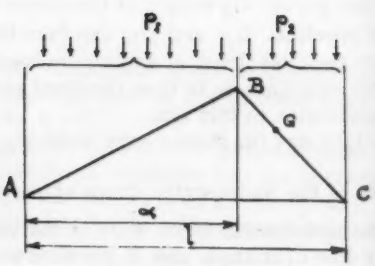


fig. 29

Action of the Internal Pressure on the Contraction Through the Poisson Effect

The effect of contraction by internal pressure may be compared to a thermal effect in which one may seek to find the equivalent variation in temperature by supposing the extremities of the pipeline to be free. Thus one finds an equivalent temperature for a pipeline with two free elbows between anchorages:

$$\Delta D = 1{,}293 \frac{r}{l_1 + l_2 + l_3} \left[\frac{l_1}{e_1} \left(\frac{p_A + p_B}{2} \right) + \frac{l_2}{e_2} \left(\frac{p_B + p_C}{2} \right) + \frac{l_3}{e_3} \left(\frac{p_C + p_d}{2} \right) \right]$$

with

	Sec. A.B	Sec. B.C	Sec. C.D
Thickness	e_1	e_2	e_3
Length	l_1	l_2	l_3

p_A pressure at A

p_B pressure at B

p_c pressure at C

p_d pressure at D

Calculation of the Reactions and the Moments of the Isostatic Arc

1. Action of one or more isolated forces:

$$v_o = \frac{1}{l} \left[\sum_A^C P (1 - \alpha) + \sum_A^C H \beta \right]$$

$$v_1 = \left(\sum_A^C P \right) - v_o$$

$$q_1 = \sum_A^C H$$

components of f_1

μ at G = the moment at G of all the forces situated to the left of G

2. Action of the vertical forces, uniformally distributed:

Let p_1, p_2, be the forces uniformally distributed per unit of projected length.

$$v_o = \frac{1}{l}\left[p_1 \alpha\left(l - \frac{\alpha}{2}\right) + p_2 \frac{(1-\alpha)^2}{2}\right]$$

$$v_1 = p_1 \alpha + p_2 (1-\alpha) - v_o$$

$$q_1 = o$$

components of f_1

μ at G = the same moment as for a straight beam on a single support.

3. Action of the horizontal forces uniformally distributed:

Let h_1 and h_2 be the forces uniformally distributed per unit of projected length perpendicular to the line AC:

$$v_o = \frac{\beta^2}{2_1}(h_1 + h_2)$$

$$v_1 = - v_o$$

$$q_1 = (h_1 + h_2)$$

components of f_1

μ at G = the moment of all the forces situated to the left of G

Calculation of the Reactions and the Moments of the Real Hyperstatic Arc

The reaction v_o is geometrically resolved at A with the force ϕ which is known in regard to its magnitude, position and direction, through its components $\mathcal{R}$, β and ρ; and at C, f_1 is resolved with the force $-\phi$.

The moment at any point G of the arc will have the value $M = \mu + \mathcal{R} + \beta x - \rho y$, μ being the value of G at the moment of the isostatic arc.

In the particular case of the fixed points, as have:

$$M_A = \mathcal{R} + \beta x_A + \varphi b$$

$$M_c = \mathcal{R} + \beta x_c + \varphi b$$

x_A and x_c being arrived at algebraically

Simplifying Hypotheses Due to the Presence of the Supporting Blocks

The presence of the supporting blocks considerably complicates the study of the pipeline considered as an arc, both by reason of the new and unknown hyperstatic factors introduced through the unilateral liaison which constitutes a support, and by the frictional forces developed at the contact with the supports and which are of uncertain value. It is also necessary to consider that the supports rest on an elastic soil, whose reactions are very largely proportional to the settlement. As a first approximation it is possible to consider the total moments at the fixed points, which have a tendency to cause a displacement of the pipeline below these supports, as without having any action.

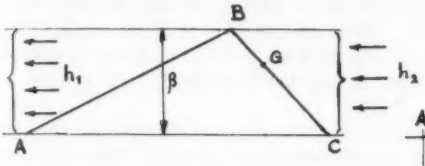


fig. 30

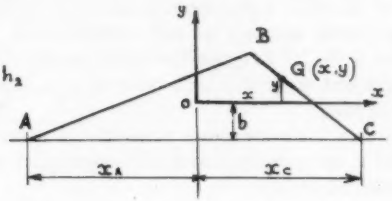


fig. 31

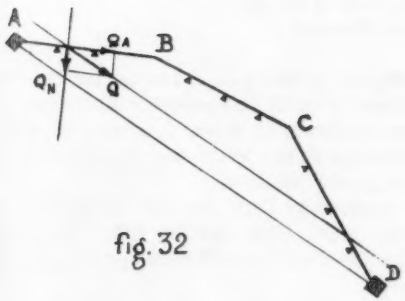


fig. 32

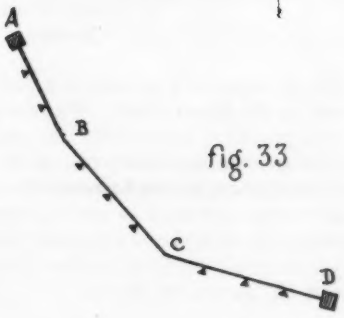


fig. 33

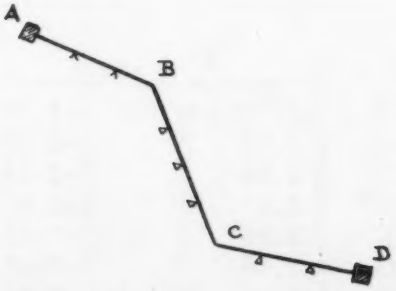


fig. 34

In the first case: Several salient elbows.

The supports may be considered as the main obstacle to thermal contraction. It may also be supposed that the thermal pressure ρ is resolved into ρ_N, normal to the pipeline and absorbed by the supports, and ρ_A along the axes of the pipeline and acting directly on AB.

In the second case: Several re-entrant elbows.

The thermal pressure due to expansion may be resolved as in the previous case, as well as the pressure due to the action of the weight and the hydrostatic forces.

In the third case: Alternate salient and re-entrant elbows.

In regard to the thermal expansion and the hydrostatic force, the elbow C and its adjacent supports may be considered as having its forces resolved in the same manner as for a fixed point. Then an analysis will be made of the arc A B C. For the thermal contraction it will be considered that B and its adjacent supports will behave as a fixed point, and the arc B C D will be analyzed.

It is, however, not possible to generalize in regard to these simplifications and it is usually found that each particular case must be made the object of a detailed study.

Practical Example Showing the Calculation of the Stability of a Pressure Pipeline Economy Resulting from a Proper Analysis of the Forces

Let us imagine a pressure pipeline whose profile and characteristics are set out in the Figure 34A. We now propose to study the possible anchorages for this pipeline, apart from the extreme anchorages A and J, which we will suppose to take the form, one of an anchorage in the rock, and the other of a solid connection to the foundations of the power station.

Let the anchorages at the elbows be denoted by C, E, and H. It will be necessary to analyze separately the arcs: ABC, CDE, EFGH, HIJ. The weight of each section, uniformly distributed, is resolved, as indicated previously at G_p and G_h as shown below:

TABLE I

	G_p	G_h
AB	3,2 t per ml	-1,06 t per ml
BC	2,68 "	-0·18 t "
CD	3,04 "	-1·14 t "
DE	2,6 "	-0·56 t "
EF	3,58 "	-1·12 t "
FG	3,48 "	-0·62 t "
GH	3,18 "	-0·24 t "
HI	3,88 "	+0·12 t "
IJ	3,96 "	-0·52 t "

The hydrostatic force at the elbows B, D, F, G, and I is resolved as follows:

TABLE II

	P	H
At B	-530 T	0
D	-490 T	0
F	-330 T	+38 T
G	-340 T	-40 T
I	+470 T	0

1° / - Study of the Filled Pipeline Under Pressure

An analysis will be made, in each arc ABC, CDE, EFGH, HIJ, of the reactions at the fixed points under the effect of the weight of the pipeline, the

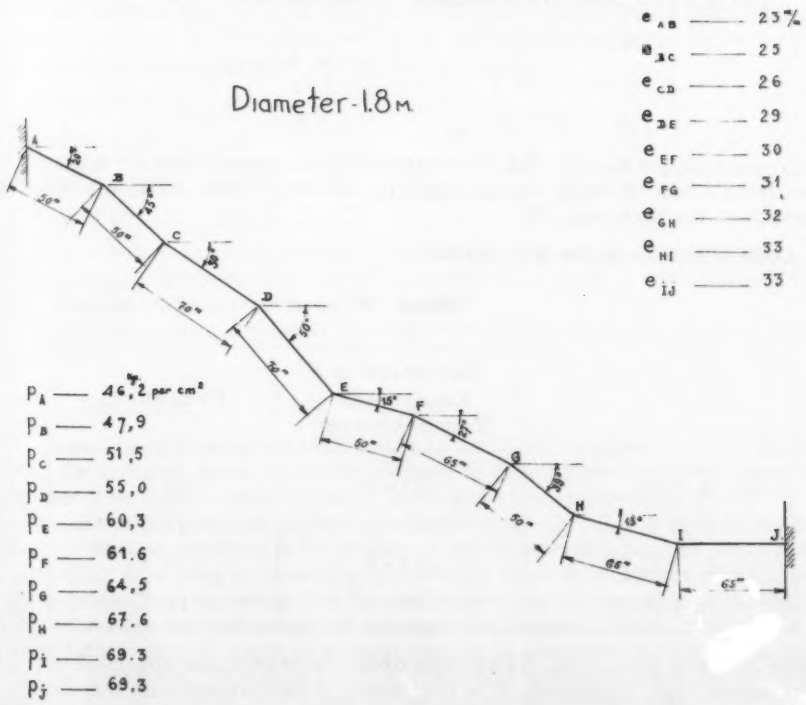


FIG 34A

hydrostatic force at the elbow, and the contraction due to the Poisson effect, and to these figures will be added the hydrostatic force on the anchorage, that is at the elbows, C, E, and H. Thus the following figures will be found:

1 - Weight and hydrostatic effects

Arc A B C

$\mathcal{A}$ = 4,100 T/M $\quad v_o$ = -112 T

β = 12 T $\quad v_1$ = -124 T

φ = -915 T $\quad q_1$ = -62 T

Arc C D E

$\mathcal{A}$ = 4,500 T/M $\quad v_o$ = -50 T

β = 18 T $\quad v_1$ = -45 T

φ = -900 T $\quad q_1$ = -119 T

Arc E F G H

$\mathcal{A}$ = 5,800 T/M $\quad v_o$ = -50 T

β = 18 T $\quad v_1$ = -56 T

φ = -800 T $\quad q_1$ = -110 T

Arc H I J (if there are no supports in this section)

$\mathcal{A}$ = 13,000 T/M $\quad v_o$ = 490 T

β = 13 T $\quad v_1$ = 490 T

φ = 2,760 T $\quad q_1$ = -26 T

In practice, for the arc HIJ, it is admissible to suppose that the supports absorb the effect of weight and of pressure, and thus there is only a negligible reaction on the anchorage H.

2 - Contraction due to the Poisson effect

TABLE III

	Variation of Equivalent Temperature	Pressure
ARC A B C . .	-23.5°	-30 T
C D E . .	-23.5°	-27 T
E F G H .	-23.8°	-31 T
H I J . .	-24.3°	-72 T

For the arcs ABC, CDE, EFGH, this pressure is partially absorbed by the supports. As, in addition, it is very small in comparison with the other forces, it may be neglected except for the arc HIJ.

In total, there will be found at each anchorage the forces set out on Fig 34B, giving a total resultant indicated in magnitude and position.

This it is possible to read immediately the moments at the fixed points, and from these figures, to deduce the corresponding stresses in the metal. Thus the following figures emerge:

TABLE IV

		MOMENT	STRESS
ARC A B C	(at A	1,110 T/M	19 kg/mm^2
	(at C	350 T/M	5,5 "
ARC C D E	(at C	1,440 T/M	22 kg/mm^2
	(at E	320 T/M	4.3 "
ARC EFGH	(at E	1,560 T/M	20 kg/mm^2
	(at H	1,430 T/M	17.5 "
ARC H I J	(at H	0	0
	(at J	0	0

2° / - Study of the Empty Pipeline

If we suppose that the pipeline is subjected to a temperature difference range of ± 35° C, the following pressures will be obtained; they are determined in magnitude and position as previously indicated:

Arc ABC : ± 44 T

Arc CDE : ± 40 T

Arc EFGH: ± 46 T

Arc HIJ : ± 103 T

In practice, due to the presence of the supports, the preponderating forces will be the pressures due to expansion for the arcs ABC, CDE, EFGH, and those due to contraction for the arc HLJ.

These pressures are small in comparison to the forces found in the filled pipeline.

3° / - Conclusion

On one hand, it can be shown that it is for filled pipelines that the stability must be ensured, since the forces developed in this case are much more important than those occurring in the case of the empty pipeline.

On the other hand, anchorages designed along the lines indicated create in their vicinities stresses in the metal, of which a certain number may appear very high; it is easy to remedy this condition by reinforcing the pipeline at these points, and by noting that these stresses rapidly diminish when the distance between the anchorage and the regarded section grows. It will thus be possible, over a length of several meters to reinforce the pipe by increasing its thickness. The stability of this pipeline is thus assured by the three blocks placed at C, E, and H and whose volume is estimated at:

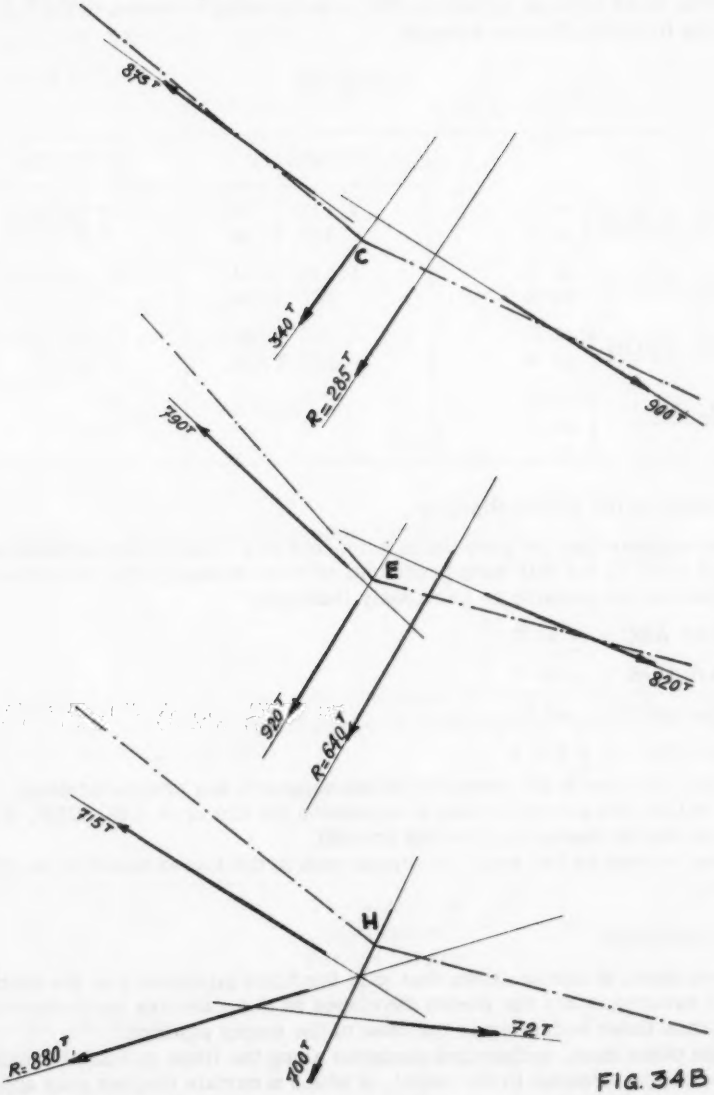


FIG. 34B

160 m^3 for the block C

330 m^3 for the block E

190 m^3 for the block H

allowing for the ground a working pressure of a maximum of 2 kg/cm^2, or a total of 680 m^3.

4° / - The Same Pipeline Designed with Expansion Joints

Let us imagine the same pipeline as before, but supposing now that it is equipped with expansion joints, and in consequence all the elbows are anchored. If we neglect the frictional forces in the joints, it may be said that the variations in temperature will have no effect on the pipeline. The preponderating forces will be the hydrostatic forces at the elbows, the resultants of which will be as follows:

At B . . . 530 T

At C . . . 340 T

At D . . . 490 T

At E . . . 920 T

At F . . . 330 T

At G . . . 340 T

At H . . . 700 T

At I . . . 470 T

The stability of this pipeline will thus be insured, in this case, by eight blocks placed at B, C, D, E, F, G, H and I, the value of which is estimated at the following figures:

630 m^3 for the block B

80 m^3 " C

860 m^3 " D

150 m^3 " E

250 m^3 " F

310 m^3 " G

120 m^3 " H

80 m^3 " I

by allowing a working pressure on the ground of a maximum of 2 kg/cm^2, and a resultant of the weight of the block and the applied forces inclined by at least 30° to the level ground; in total, the volume of concrete is some 2,480 m^3.

5° / - Summary

This example thus shows the advantages of a pipeline without joints, with free elbows. On one hand, the pipeline with joints requires a volume of material for the anchorages nearly four times greater than that for a pipeline

without joints; in addition, the joints themselves require considerable cost and labor for construction and maintenance. On the other hand, the careful design of the anchoring points of the pipeline without joints, as well as an exhaustive study of the forces developed in the pipeline, will allow the project to be carried out economically by means of an integrated arrangement of pipe and anchorage where the concrete and the steel are solidly joined together, as well as with the reinforcement of several points where the stresses in the steel appear to be too high. The increased price required by reason of this reinforcement bears no comparison to the economy achieved by the reduction in the volume of the concrete.

Calculation, as a Practical Example, of the Stability of an Anchoring Block Economy Resulting From a Correct Analysis of the Forces

Introduction

The standard anchoring blocks are constructed of concrete or stonework, and these works, by reason of the combination of their weight and their reactions with the ground on which they rest, are able to provide a state of equilibrium in regard to the forces which are transmitted to them by the pipeline.

As the anchoring blocks are ordinarily buried in the ground, the problem of their stability transforms itself into a problem relating to the resistance of the ground, and in this problem there is imposed either a predetermined working load or a given displacement. The limiting values of these parameters will depend to a considerable extent on the terrain under consideration, and to a lesser degree on the geometrical shape of the foundation.

It is not within the scope of these notes to discuss the various theories and methods employed in the determination of the reactions of the terrain, as these matters relate more particularly to the field of soil mechanics. It may however be mentioned that the variation in the elasticity of the soil as a function of its depth, the prestressing of the soil, and the state of the fluid phase must all be taken into account. Thus a "foundation coefficient" may be defined, which represents the stress causing a unit measurement of sinkage into the soil; and besides depending on the factors already mentioned, this coefficient will also be influenced by its form and the surface of the loaded portions.

Account must also be taken of the friction of the block on the walls and the floor, and the effect of the thrust of the block on the soil.

Calculation of this type involves difficulties and uncertainties, particularly in regard to the determination of the thrust of the soil, and in relation to the friction between the concrete and the ground. It is in general preferable to make the following assumptions:

1° / - To consider a projected surface of the buried contour of the block, perpendicular to the resultant of the applied force and the weight of the block.

2° / - To assume a linear distribution of the varying pressures from one edge of the block to the other.

3° / - To consider, as a limiting value for the resistance of the soil, a global value determined once and for all by tests of the soil on site.

Mention should also be made of the fact that certain blocks, (for example, those on the Colorado Big Thompson project), which are founded on rock or on normal ground have been calculated in regard to frictional resistance for a coefficient of 0.65 on rock and of 0.35 to 0.50 on ordinary soil, according to the site conditions.

Stability Diagram of a Gravity-Type Block

A - Case of a rectangular supporting surface
C is the meeting point of the resultant and the base
G is the center of A - B

1st Case C is found on the axis of symmetry of the base rectangle, in the central third.

The maximum stress at B and the minimum at A will be:

$$n = \frac{R}{l\ b} \quad (1 \pm \frac{6\ e}{l})$$

2nd Case C is found on the axis of symmetry of the base rectangle, outside the central third.

The total force is distributed over the compressed portion since the ground is not capable of resisting tractive forces.

The maximum stress at B will be:

$$n = \frac{2\ R}{3\ b\ (\frac{1}{2} - e)}$$

3rd Case C is not found to be on the axis of symmetry of the base rectangle.

The line E C D is then drawn so that C is at the center of ED, and two forces, each equal to $\frac{R}{2}$ are applied at E and D. Separate calculations are then made, as seen previously, of the maximum stresses developed by each of these forces, and the effects produced are then superimposed.

B - Case of a nonrectangular supporting surface

The central inertia ellipse of the projection of the contour of the block is constructed. If C is the meeting point of the resultant with this surface, the stress at a point a will be:

$$n = \frac{N}{\Omega} \quad (1 + \frac{z_o z}{\rho_z^2})$$

in which:

z_o = GC

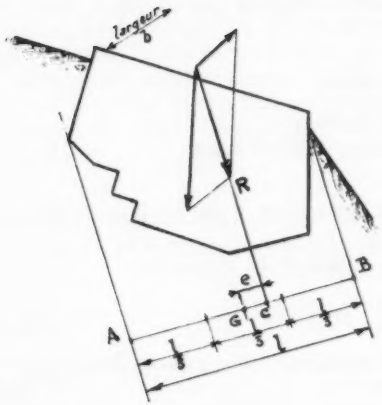
z = b a

p_z = GE

N = applied forces

Ω = surface of the projection

Gy = conjugate diameter CG in the inertia ellipse



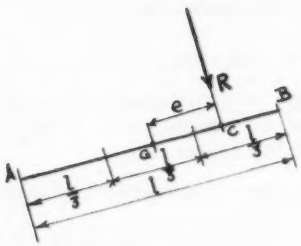


fig. 35

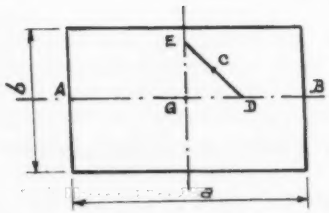


fig. 36

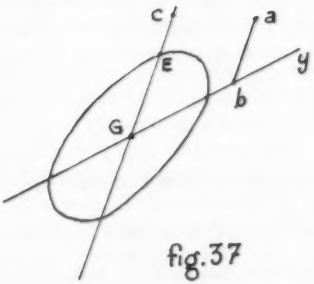


fig. 37

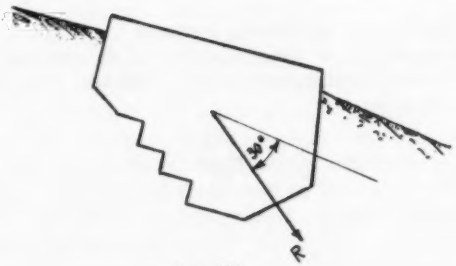


fig. 38

Stability Conditions

Direction of the resultant of the weight and the applied forces.

This resultant must set up an angle, with the ground situated on the downstream side of the block, which is sufficient to avoid slipping of the soil under the effect of the pressure. For an average type of soil, the usual angle is 30°.

Maximum Stress

The maximum stress, calculated on the lines of the preceding section indicated by, should not exceed the characteristic value for each type of ground: if this value has not been determined by site tests on the soil, the following figures may be used in the preliminary planning stage:

TABLE V

	Nature of the Terrain		Admissible Working Load in Kg/cm^2
Compressible Soil	Humid or water-bearing	(Silt (Swamps (Ground with running water	0 to 0.1
	Coherent soil, dry or slightly humid	(Embankments or pushed-back earth, not rammed	0 to 1
		(Vegetable-growing soil	0.5 to 1
		(Clayey soil (soft	0.4 to 0.8
		(Clayey soil (sound	0.8 to 1.5
		(Slightly sandy clay) semi-hard consistency (Earthy gravel) semi-hard consistency	1.5 to 2.5
		(Sandy clay) hard consistency (Compact clay) hard consistency	2.5 to 3.5

TABLE VI

	Nature of the Terrain		Admissible Working Load in Kg/cm^2
Soil Only Slightly Compressible or Noncompressible	Noncoherent soil, but soil of high compactibility	(Fine and average sans ($\lesssim$ 1 mm	2 to 2.5
		(Coarse sand 1 - 3 mm	3 to 4
		(Gravelly sand (33% grains of ($\lesssim$ 70 mm)	4 to 5
		(Pebbly gravel	4 to 7
	In stratified layers	(Soft rocks (schists, calcareous (rocks)	7 to 10
		(Semi-hard rocks (sandstone, (limestone, marble, dolomite)	15
	In solid layers	(Hard rocks (granite, diorite, (porphyry, diabasic, basalt, (gneiss, quartzite)	25 to 40

Example of the Economy Realized by an Improved Degree of Determination of the Operating Forces

In the example taken in Chapter IV we discussed a pipeline with free elbows supported at A, G, E, H and J. Let us consider the block situated at elbow E.

A brief and elementary analysis of the operating forces will give the following results:

1. When the pipeline is empty and is subjected to a decrease of temperature of -35° C with relation to the temperature at which the joints were made, it might be thought, due to incorrect reasoning, that the thermal contraction sets up an axial force (in the same way as for a straight portion of the pipeline comprised between two anchorages) equal to $0.232 \Delta D \times S$ (see Section 1), which would amount to 1,330 T for the section ED, and 1,380 T for the section EF. The resolution of these two forces will give a resultant of 820 T.

2. When the pipeline is full the analysis may be made in two ways:

a) The hydraulic force on the elbow E is as we have seen, 920 T. By the same reasoning as for the thermal contraction, it might be thought that the contraction due to the Poisson effect would set up an axial force equal to $0.471\, p_m\, D^2$ (see Section 1), which would amount to 880 T for the section ED and 930 T for the section EF, giving a resultant of 550 T which combined with the figure of 920 T would give a total resultant of 400 T.

fig.39

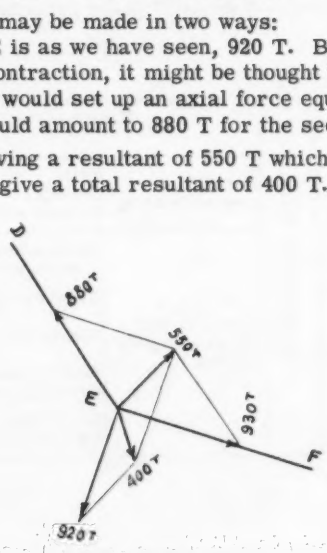


fig.40

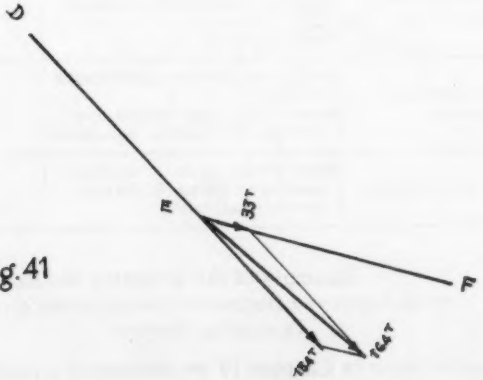


fig.41

The preponderating force will thus be due to the thermal contraction of 820 T for the empty pipeline, which will require an enormous block at this point.

b) It may also be considered that at E there operates in the direction ED the difference of pressure between D and E, and, in the direction EF, the difference of pressure between E and F. One will thus obtain a total hydraulic force of 164 T, the resultant of 134 T for the section DE and 33 T for the section EF.

No account is taken of the Poisson effect, and it could be stated that the preponderating force would be that due to the thermal contraction which would amount to 820 T.

According to these two methods of reasoning, one is led to design the dimensions of the block E to resist an outgoing force of 820 T, which will result in a block of 800 to 1,000 m^3 in place of the block 330 m^3 indicated in Section IV. Thus it will be noted how greatly over-dimensioned the block will be if it is designed according to a superficial and erroneous analysis of the forces acting on the anchorage.

Example of the Economy Which may be Realized by a Better Design of the Block

We now propose to calculate the anchoring block of the downstream extremity of a pipeline. The pipeline has a diameter of 2 m, and is provided at its downstream end with a valve, which sets up, when it is closed, an axial pressure of 500 T.

A maximum unit pressure of 1.4 kg/cm^2 is taken for the soil, and it is also assumed that the resultant makes a minimum angle with the soil of 30°.

First Solution - Solid Concrete Block

This block will have the shape indicated in the adjoining diagram with a length of 18 m, a width of 5 m, and a total volume of 340 m^3.

Second Solution - Hollow Block in Reinforced Concrete

It is possible to consider reducing the volume of concrete used in the preceding block; the concrete itself is, in effect, one of the most costly elements in the civil engineering aspect of a pressure pipeline.

For this purpose, the block has been conceived in the following manner: the designer has imagined a block of reinforced concrete with three hollow caissons filled with rammed earth, and two lateral walls receive the axial pressure of 500 T through the intermediary of a diaphragm of reinforced concrete on which the pipeline is fixed, and convey it to an oblique soleplate. The block is entirely buried, and the total weight, which combines with the axial pressure of 500 T, results on one hand from the weight of the concrete, and on the other hand from the weight of the internal and external filling. Thus one obtains a volume of concrete of 200 m^3, with a distribution of weight of 480 T for the concrete, and 140 T for the filling.

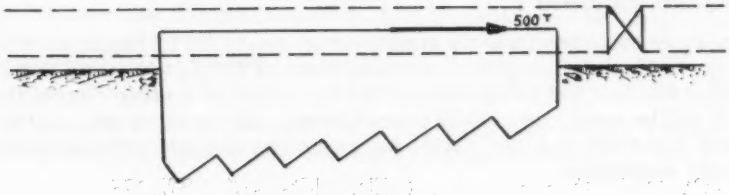
The economy of 140 m^3 of concrete thus realized, when the latter solution is compared with a solid block, largely compensates for the additional shuttering and the steel necessary for the reinforced concrete. The earth work needed for the foundation of these two blocks is approximately the same.

Conclusion

We hope thus to have shown that a closer determination of the operating forces may lead to a reduced dimension, and that in the range of forces thus determined, it is possible to design anchoring systems lighter than those which relate to compact stonework designs.

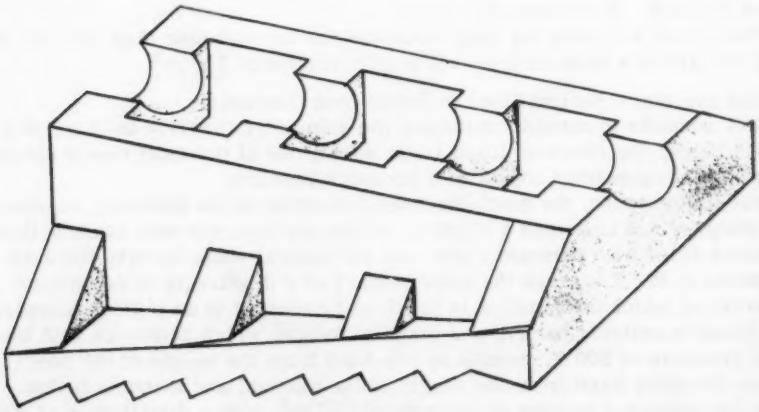
Stone Anchorages

The forces acting on the anchoring blocks of pressure pipelines may in general be resolved into two forces, one normal to the ground and the other



Solid Foundation

fig. 42



Hollow Foundation

(Filled with tamped soil
Block entirely buried)

parallel to the surface of the soil. When the normal component tends to separate the block from the ground, the designer seeks to evolve a block of such a shape and weight that the resultant of the weight and the applied force is directed as follows:

1°) - In such a way as to apply the block into the ground,

2°) - In such a way as to avoid all sliding action of the ground on itself,

3°) - In such a way as not to exceed the admissible pressure on the ground. It follows that blocks of this type become very large. In addition, the designer has sought to compensate the operating force by means of a directly opposed reaction, in the form of a type of anchorage relaying the effect of the force into the surrounding ground.

In a salient elbow, the force which tends to separate the elbow from the ground may be resolved into as many components F as there are elements in the elbow. Any such component F may be absorbed by two anchorages placed at either side of the pipeline, and each absorbing $\frac{F}{2}$. These anchorages may be made up in the form of rods or cables. Two cases may be discussed, according to whether anchorages are made in rock, or in ground without rock.

Anchorages in Rock

If the sound rock shows above the ground, the problem consists in calculating the length of the rod or cable necessary to obtain, in regard to fastening down, a sufficiently high adherence between steel and concrete, and between concrete and rock. The coefficient of safety to be applied will take account, in particular, of the nature of the rock, the inclination of the layers of rock, the method of fastening, and the alternation of the applied forces. If the sound rock exists at a depth of several meters below the natural soil, the same calculation may be applied in regard to the part of the rod or the cable which is sealed into the rock. The part situated in the loose soil may be sheathed while drilling and coated with concrete. As for the part in the rock, this concrete is being provided only to protect the metal against corrosion.

Stone Anchorages in Ground Without Rock

When the ground does not contain rock, it is not possible to find, in the adherence between concrete and rock, the compensating force in regard to the applied force. By analogy with the foundation pile which, by the operation of the thrust and the friction on the ground, resists forces directed from the top downwards, the designers have studied the behavior of a pile subjected to forces directed from below upwards, which tend in consequence to draw the pile out of the ground.

The type of pile considered is one which is cast in the ground with an exploded bulb. It includes along its axis a rod or cable which is spread out in the bulb at the base. The problems which arise in connection with an anchorage of this type for a pressure pipeline are as follows:

- Study of the elongation under a given load, as a function of time,
- Variation of the elongation with the applied tractive effort,
- Behavior of the pipe under alternating forces; stabilization of the pile,
- Practical development of this type of anchorage.

We have for this purpose, carried out a series of tests on small-scale models, as well as a further series of tests on full-scale equipment.

I - Tests on small-scale models

These models are made up of steel rods 400 to 800 mm in length, 14 mm in diameter, terminated by a cone whose diameter can vary between 36 and 59 mm. These rods are placed in a rammed sand, lightly humidified. An increasing tractive effort is exercised on the ends of the rods, each new traction test being added to the preceding test.

Elongation under given tractive effort as a function of time:

The curves in Fig. 47 show the variation in the elongation as a function of

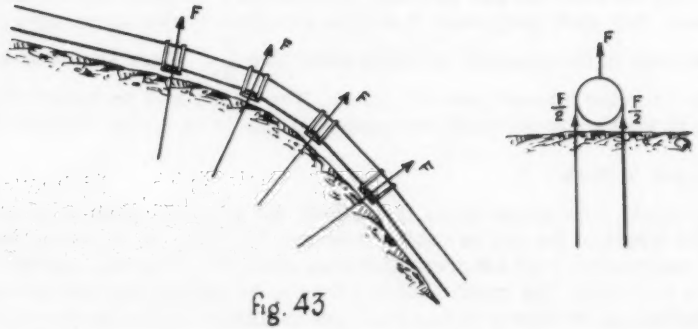


Fig. 43

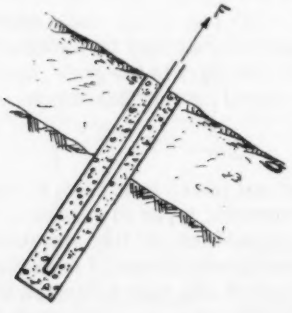


fig. 44

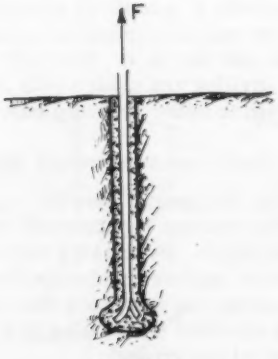


fig. 45

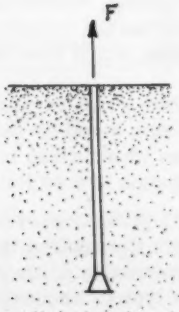


fig. 46

time, under various tractive efforts. For the lower tractive efforts, these elongations tend towards a limiting figure. For higher tractive efforts, the limit becomes less precise, and a time arrives when the elongations are proportional to time, and this is exactly in accordance with the condition of drawing-out.

Elongation as a function of the applied tractive effort:

Fig. 48 gives an example of the type of elongation (measured at the end of 15 minutes after the application of each load), for tractive efforts varying between 0 and 240 kgs. This curve is deformed if the elongations are measured at the end of a time period differing from that mentioned above; it is in itself insufficient to represent the phenomenon, and must be accompanied by curves showing elongation as a function of time which will show if for a given tractive effort, there does or does not exist an elongation limit. The drawing-out force of the pile is that for which it is no longer possible to find any such limit.

Behavior of the pile under alternating loads:

At the site of an elbow on a pressure pipeline, variable forces may manifest themselves, and these may be more or less severe. Water hammer brings in its train variations of hydraulic force, and there are also variable

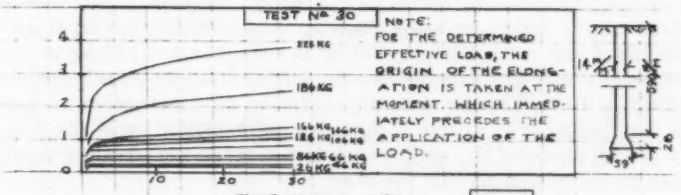

TEST Nº 30
NOTE:
FOR THE DETERMINED EFFECTIVE LOAD, THE ORIGIN OF THE ELONGATION IS TAKEN AT THE MOMENT WHICH IMMEDIATELY PRECEDES THE APPLICATION OF THE LOAD.
TIME IN MINUTES

FIG. 47

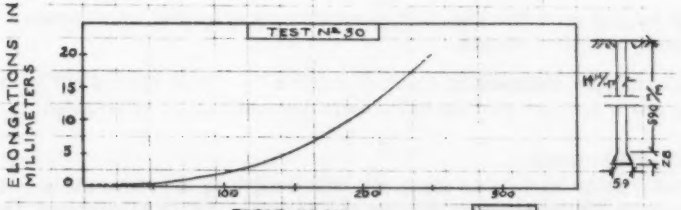


FIG. 48

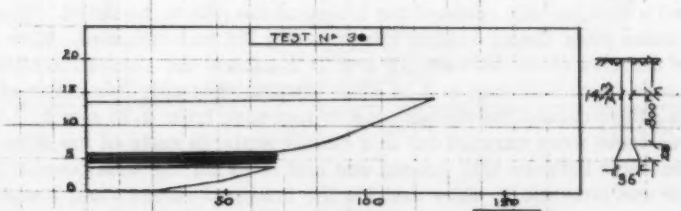


FIG. 49

contractions and expansions arising from thermal causes. These forces are translated into variations of the forces on the anchorages, and inquiry must be made as to whether the stability of the pile under a load T does not become modified when the force varies more or less in relation to this value.

Fig. 49 shows the elongation curve of a pile under the following loading conditions. A tractive effort, regularly increasing from 0 to 66 kg was applied, and the elongation under an affort of 66 kg was 3.8 mm. Then the load was brought back to zero. It was immediately noted that the system did not revert to its initial state, since the soil had been disturbed in an irreversible manner. Next, a series of loading and unloading cycles, from 0 to 66 kg were applied, and it was then seen that the elongations increased by an amount which grew progressively smaller.

Fig. 50 shows the elongation curves of a pile of the same type under the following loading conditions: the same number of alternations of load, of the same amplitude (40 kg) were applied, but were carried out from different loads (66 kg, 106 kg and 146 kg). The tractive effort was not brought back to zero at any time. In Fig. 51 the elongations obtained are plotted as a function of the number of alternations. These two associated graphs enable the existence of a limit of elongation as a function of the amplitude of alternations, the number of alternations, and the tractive effort from which they are exercised could be obtained.

In Fig. 52, alternations of tractive effort on the same amplitude as before were carried out, from the same starting load (106 kg), but first a greater tractive effort had been applied; 126 kg in one case and 146 kg in another case. It was shown that the greater the initial tractive effort, the smaller the total elongation between the start and finish of the alternative load cycle.

It is thus possible to create a condition of stabilization of the pile, resulting in a stabilization of the soil, which is the more efficient the more the following three conditions are realized:

1. The initial tractive effort on the pile has a value greater than the utilization tractive effort,

2. A decrease in the residual elongation is achieved by a series of alternating tractive efforts,

3. During the alternating tractive efforts the force should not be brought back to zero, and the pile should always be maintained in tension.

II - Full-scale tests

Various piles have been made up, with length of approximately 5 m, made up of a cable embedded in a column of concrete cast in the ground, the base of this column being expanded into a bulb. A system of jacks enabled the application of a known degree of tractive effort to be provided at the head of the pile, and a comparator enabled the lifting of the pile to be noted. Thus for one of these piles the elongation curve in Fig. 53 was obtained. Five alternations of tractive effort between 15 and 20 T caused the elongation of the pile to increase from 2.23 mm to 2.31 mm. For another pile five alternations of tractive effort caused the elongations to increase from 1.73 mm to 1.83 mm.

These tests were carried out in a clayey soil. In spite of the difference of characteristic between this ground and that used for the small-scale model tests; it was possible to show that for the full-scale tests also, a stabilization of the pile was reached after the application of alternating forces. The

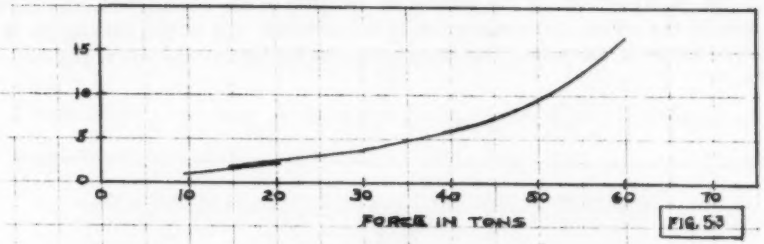


FIG. 53

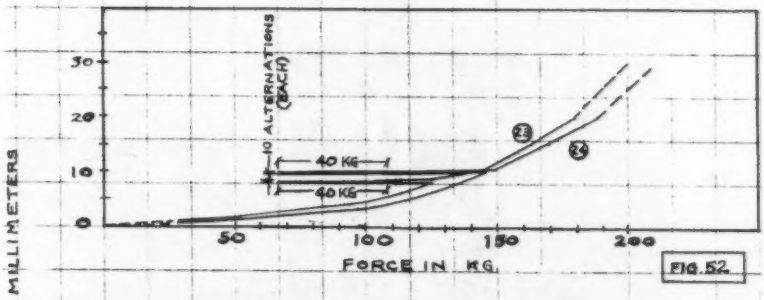


FIG. 52

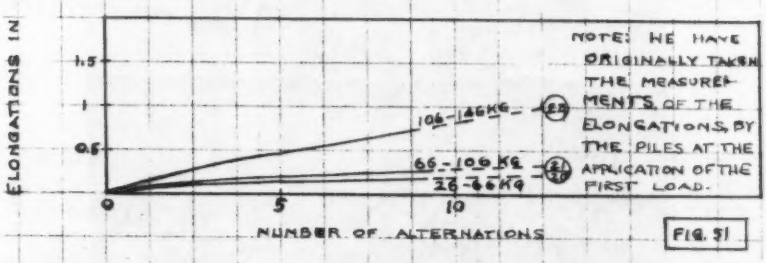


FIG. 51

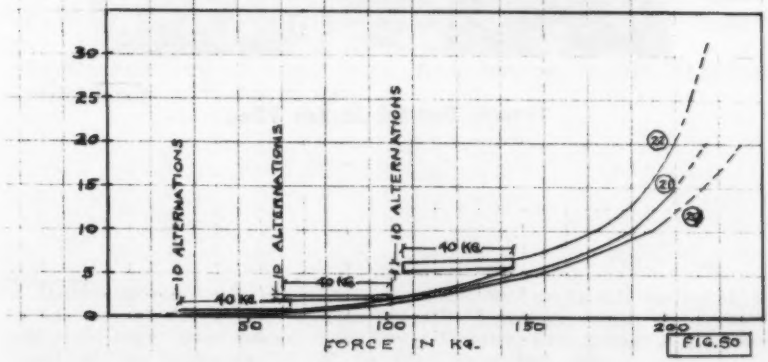
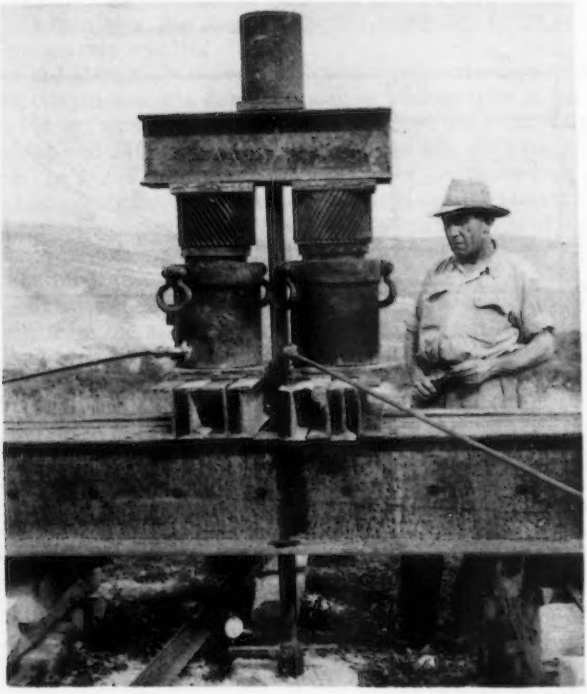


FIG. 50

interest of these tests resides also in the possibilities of development of piles of this type, which provide problems of drilling, of the carrying out of the exploded bulb portion, of the setting in position of the completed job, and the guiding of the cable, the concreting of the column, and of the setting up of the tractive force in the pile. The photograph below shows the arrangement of the tests.



Tensile Test for Anchor Piles

Journal of the
POWER DIVISION
Proceedings of the American Society of Civil Engineers

AN UNUSUAL DEPOSIT IN A HYDRAULIC TUNNEL

Arthur Lewis Pollard[1] and Hazen Etnier House[2]

SUMMARY

Analysis has shown that the rough black deposit adhering to the walls of this power conduit consists of metallic oxides, the most abundant of which is manganese.

This deposit can cause an increase in annual power loss of 5,000,000 kwh. The process by which the materials are extracted from the lake water and deposited on the walls has been investigated. To date, the most effective means of removal is mechanical cleaning by hydraulic jets.

INTRODUCTION

The Santeetlah Hydroelectric Project now owned by Tapoco Incorporated, a fully owned subsidiary of ALCOA, was first placed in operation in 1928.

It is located in Graham County, Western North Carolina. The 212 ft. high dam is located on the Cheoah River and supplies water through an 11 ft. diameter pressure conduit approximately 4.75 miles long to the power house situated upstream on the lake formed by Cheoah dam on the Little Tennessee River.

The excessive hydraulic head losses first observed in 1945 led to the investigation described in this paper.

Note: Discussion open until May 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2312 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 6, December, 1959.

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Description of Santeetlah Pressure Conduit

Santeetlah Hydroelectric Plant is supplied by a closed pressure conduit eleven feet in diameter and 24,110 feet long, consisting of 16,537 feet of concrete lined tunnels and 8,573 feet of riveted steel pipe. The maximum gross head with a full lake is 663 feet. At a plant output of 40,000 kw approximately 900 cfs water is required amounting to 582 million gallons per day.

When the plant was first placed in operation in 1928 a test was run to determine conduit friction head losses, which were found to be 40.6 feet at 900 cfs. During a turbine test made in March 1945, it was discovered the conduit head loss had risen to 55 feet at 900 cfs.

Inspection of the conduit in 1945 showed no structural damage to the concrete lining and the steel pipe which was painted with bituminous paint in 1939 was found to be in excellent condition. However, a rough black fouling deposit existed, on the concrete tunnel linings, that was in some cases 5/8 inch thick with an average of 1/2 inch depth overall.

A detailed head loss test was run in 1956 and friction coefficients for the concrete and steel sections were accurately determined and found to be:

Concrete tunnels $n = 0.0162$

Riveted steel pipe $n = 0.0142$

where "n" is the coefficient in the Manning formula

$$n = \frac{0.4631\ d^{8/3}\ s^{1/2}}{Q}$$

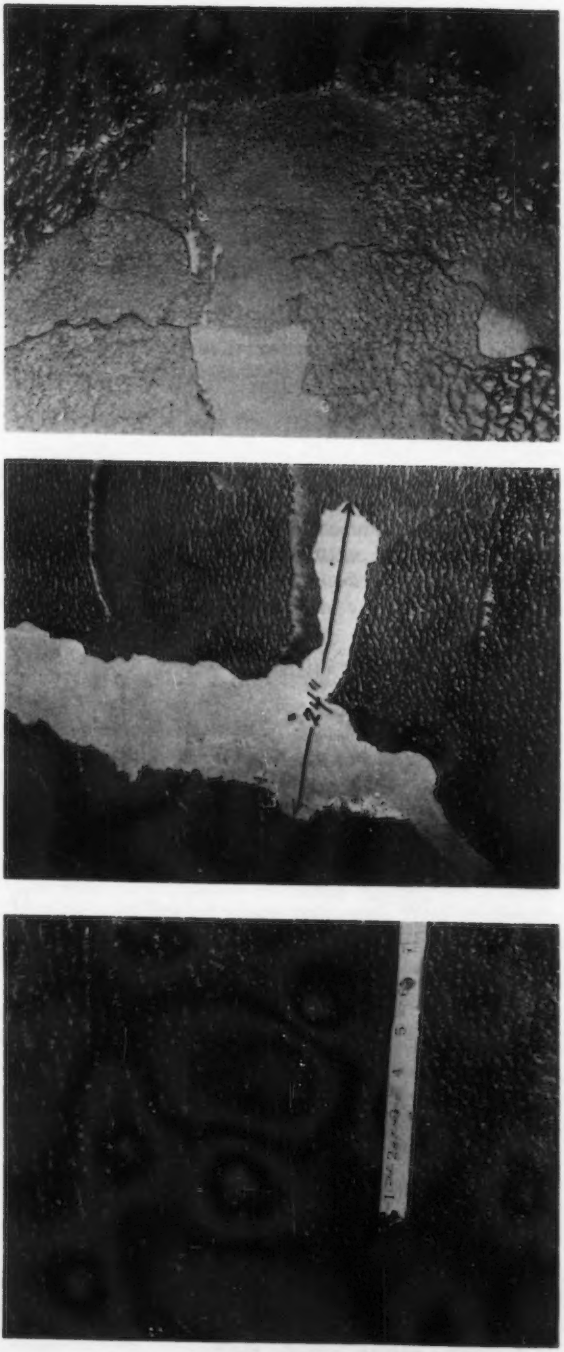
and d is diameter in feet, Q discharge in cubic feet per second, and s is loss of head caused by friction per foot length of pipe.

Based on test data from many observers, the concrete tunnels should have a lower friction coefficient than the riveted pipe. Riveted steel varies from 0.013 to 0.017 and concrete 0.010 to 0.014.

Assuming loss coefficients for the bends, steel pipe, and entrance losses have remained unchanged, "n" for the concrete lined tunnels would be 0.0109 to give a total conduit loss of 40.6 feet at 900 cfs when the plant was new. Reduction in tunnel diameter of 1 inch by the fouling deposit would account for only 0.8 foot additional head loss leaving 13.6 feet increase caused by the roughness of the deposit.

Physical and Chemical Nature of the Deposit

The deposit on the walls of the concrete section of the tunnel has a maximum thickness of 5/8". In appearance it is something like a dense layer of soot. Fig. 1 shows the deposit on the concrete lined tunnel section in pictures taken July 8, 1949. Upon drying, it tends to become friable and easily broken into small flakes. A section of the deposit when mounted in paraffin and examined under a wide field microscope shows a lameller structure. The first 4 layers next to the walls of the tunnel are widely variable and probably represent a period of about 4 to 8 years during which the mineral and biological character of the lake approached equilibrium. The total number of layers vary considerably with different samples but the best specimans examined



(a) (b) (c)

Fig. 1. Fouling Deposit in Santeetlah Conduit, Yellow Mountain Concrete Lined Tunnel, Near Portal No. 8 - July 15, 1949.

show 18 layers probably corresponding to yearly accumulations. The first layer is black, 0.6 mm in thickness, and seems to be of the same character as later deposits. The second is red, 0.5 mm, and consists predominantly of iron. The third is black, 0.9 mm, of the same character as the first and later deposits. The fourth is red, 1.0 mm in thickness, and is the thickest and most well defined of any of the layers. It is difficult to explain this later unless it represents a year of active rusting of the interior of the steel penstocks. The 14 subsequent layers are all black and vary in thickness from sample to sample, but average about 0.9 mm in thickness. These undoubtedly represent equilibrium conditions and are obviously laid down at yearly intervals.

Chemical analyses were made by the Alcoa Smelting Plant Laboratory in March 1948 and July 1951. These are shown in Table 1. These analyses demonstrate the sponge-like nature of the deposit. On the basis of 1 kg of deposit, the distribution of various classes of material is as follows:

Water and volatile organic matter	870 grams
Organic matrix	25 grams
Mineral Matter	105 grams
	1000 grams

An analyses of these figures shows that the organic matrix is about what would be expected from the composition of plant material. This residual matrix is predominantly of a polysaccharide character; the nitrogenous material having been almost completely utilized by saprophytic* microorganisms.

TABLE I

ANALYSIS OF TUNNEL DEPOSIT

Alcoa Smelting Plant Laboratory	March 1948	July 1951
Loss on heating at 105°C	85.6%	90.0%
Analysis of dried solids		
Loss on ignition 700°C	18.6	19.3
Ash	81.4	80.7
Analysis of ash		
Si - as SiO_2	7.0	11.8
Fe - as Fe_2O_3	18.5	7.9
Al - as Al_2O_3	7.5	
Ca - as CaO	1.0	5.6
Mg - as MgO	2.5	
Mn - as Mn_2O_3	59.5	56.3

* Organisms which live on rotten organic matter.

The mineral matter, however, is from 2 to 3 times what would be expected. This excess mineral matter is of two general types.

A - Entrapped rock fragments consisting of silica and alumina.

B - Iron and manganese oxides adsorbed on the polysaccharide.**

The entrapped rock fragments account for about 25% of the excess mineral matter. The balance is predominantly manganese oxide with some iron oxides.

Analysis of freshly deposited material on a microscope slide to determine the character of the organic matter showed that it is largely polysaccharide. Microscopic examination of these slides confirmed this since the deposit had a typical appearance of decomposed plant material: that is, of gelatinous masses with vestiges of recticular structure.

Preliminary Investigations of the Source of the Deposit

The fouling of water pipes by the filamentous iron and manganese bacteria (Leptothrix) has been often described, Ellis (1916)(1) and Harder (1919)(2) and Pringsheim (1949),(3) and it was first assumed that this must be the cause of the deposit. A great number of microscopic examinations of the deposit were made, but only a very few structures which might be Leptothrix were found. Some 80 attempts were made to cultivate these organisms from the deposit and from the lake water. These failed completely although all recommended media were tried. It was finally concluded that organisms of this type could not be the cause of the fouling.

By-pass cells containing microscope slides were next installed at Calderwood power house, Santeetlah dam and Santeetlah power house. These slides were examined at monthly intervals for several months. They showed the gelatinous organic matter found in places of the deposit along with numerous dead diatoms among which were species of Tabellaria, Synedra, Navicula, Cymatopleura, Ceratoneis, Fragilaria, Cymbella and great quantities of Amphora. Dead blue-green algae were also found among which were Gloeocapsa, Osciliatoria and Spirulina. Strangely, the siliceous diatom shells were not found frequently in the older tunnel deposit. As the layers built up during certain parts of the year these shells probably washed away from the gelatinous material. The slides examined under polarized light showed numerous particles of crystalline rock fragments probably washed into the lake from roads and cultivated land and subsequently trapped in the mucilaginous organic matter.

Scrapings from these slides were treated with gluconic acid to remove metals. They were then washed with 6N HCl and analyzed for polysaccharides with the anthrone reagent. Results were very positive. Solubility tests, using graded concentrations of H_2SO_4, indicated that the polysaccharides were mixtures of plant materials ranging from pectins to cutins. These scrapings also contained widely varying quantities of manganese.

Model Experiment to Determine Extent of Manganese Adsorption on Polysaccharides

Since preliminary work indicated the presence of polysaccharides in the deposit, it was decided to study manganese adsorption in a controlled system.

** Substances such as starch, cellulose, pectin, etc.

This study was published in SCIENCE, Pollard and Smith, 1951.[4] It was found that adsorption of metals is a quite general property of polysaccharides. This property is dramatically illustrated by the discovery of mineralized trees by uranium prospectors.

Source and Nature of the Manganese in the Lake Water

During 1951, a series of analyses were made on the lake water and on the 8 feeder streams. In some of these determinations the water was well aereated and allowed to settle for 7 days; manganese was then determined separately in the supernatent and sediment.

Typical ranges were:

pH by bramine thymol blue	6.1 - 7.0
Mn total	0.005 - 0.03 ppm
Mn free from sediment	0.000 - 0.005 ppm

Total manganese roughly approximates the amount of sediment.

It will be noticed from the spectrographic analysis, Table 2, that the sediment contains about half as much manganese as iron, yet we have the remarkable fact that the tunnel deposit contains from 4 to 6 times as much manganese as iron. It is apparent that iron is exchanged for manganese at the surface of the tunnel deposit.

Postulated Mechanism of Deposit Formation

The deposit shows clear evidence of formation in annual layers, and it is known that the principal accumulation is during the fall turnover when decomposing organic matter rises to the surface. This material is largely polysaccharide in nature. It consists of cellulose, chitin and cutin with other unknown substances. Some of it is from organisms growing in the lake and some is washed in from the forest floor, In the lake bottom it apparently undergoes an aerobic formation. As the particles rise during the fall, the iron and possibly the manganese partly oxidize. In this condition they are mucilaginous and adhere strongly to any surface. At this stage, the new deposit contains more iron than manganese.

TABLE II

QUALTATIVE SPECTROGRAPHIC ANALYSIS OF ASHED SEDIMENT

University of Tennessee Spectrographic Laboratory, October 1951

Biologically Important Metals

Cobalt	Less than	0.008%
Molybdenum	Less than	0.008
Iron	More than	1.3
Magnesium	Less than	6.58
Manganese	Approximately	0.66
Copper	Less than	0.66
Calcium	More than	6.58

The freshly deposited particles are seen on the older black deposit as ochreous blobs. In inspecting the drained tunnel, these appear in all shades from the decidedly light brown of the freshly deposited to the almost black of the older deposit. This change has been duplicated in the Laboratory. It appears to be selective adsorption of manganese and desorption of iron.

Effects of Calcium Ions

It is well known that calcium ions have a hardening effect on gelatinous polysaccharides. This led us to make a number of laboratory experiments on the water from the alke and feeder streams. This water contains a non-colloidal sediment fraction which settles quickly and a colloidal fraction which remains suspended for some time. We found that the addition of as little as 1 ppm of calcium ions in the form of hydroxide caused much more rapid settling of the colloidal fraction.

Preliminary experiments had been tried at the time of filling the tunnel after the annual inspection. During the refilling, calcium hydroxide was added at the inlet to give a final concentration of 20 ppm. When the turbines were started an impressive amount of the deposit was washed out.

Continuous Feeding of Calcium

Continuous feeding of calcium hydroxide at the rate of 1 ppm was tried during the fall of 1954. Unfortunately it was started on October 15th which proved to be late for the conditions that year and the results were inconclusive. The year was unusual due to the low water level which persisted for several months and the turnover evidently started much earlier than previous records led us to expect. In spite of this there was no increase in head loss during the feeding period which extended from October 15 to November 15.

During 1954, a number of new analytical procedures were started in which iron analyses of the water entering and leaving the tunnel were made at frequent intervals. At the same time the sediment was examined on millipore filters. It is apparent from Fig. 2 that the iron content of the entering water correlates with the head loss.

CONCLUSIONS

Head loss caused by the tunnel deposit represents a serious economic loss. The obvious solution is mechanical cleaning and this was done in August 1951 and again in August and September 1956. The work was done with portable pumps, aluminum irrigation pipe and hydraulic jets. This reduced the head loss about 20 feet at 900 cfs. The cost of $22,650 was reasonable, but the method involved closing down the plant for a number of days. This however was at a time when the reservoir was very low.

At the time when the deposit was supposed to be caused by living organisms, continuous chlorination was suggested. Since the deposit is now known to be caused by dead organic matter, such treatment is obviously useless. Lime feeding was the only other treatment which seemed at all feasible. However, the lime feeding during September, October, and November 1955 cost $3,900 with only a small reduction in head loss. It appears from the data

available that the most effective method of reducing the head loss caused by the fouling deposit is mechanical cleaning every five years at an annual average cost of about $4500 per year.

The authors gratefully acknowledge the assistance of Dr. Isabel H. Tipton and Mr. Anatole Kotloby of the Physics Department of the University of Tennessee for spectrographic analysis of ashed sediment.

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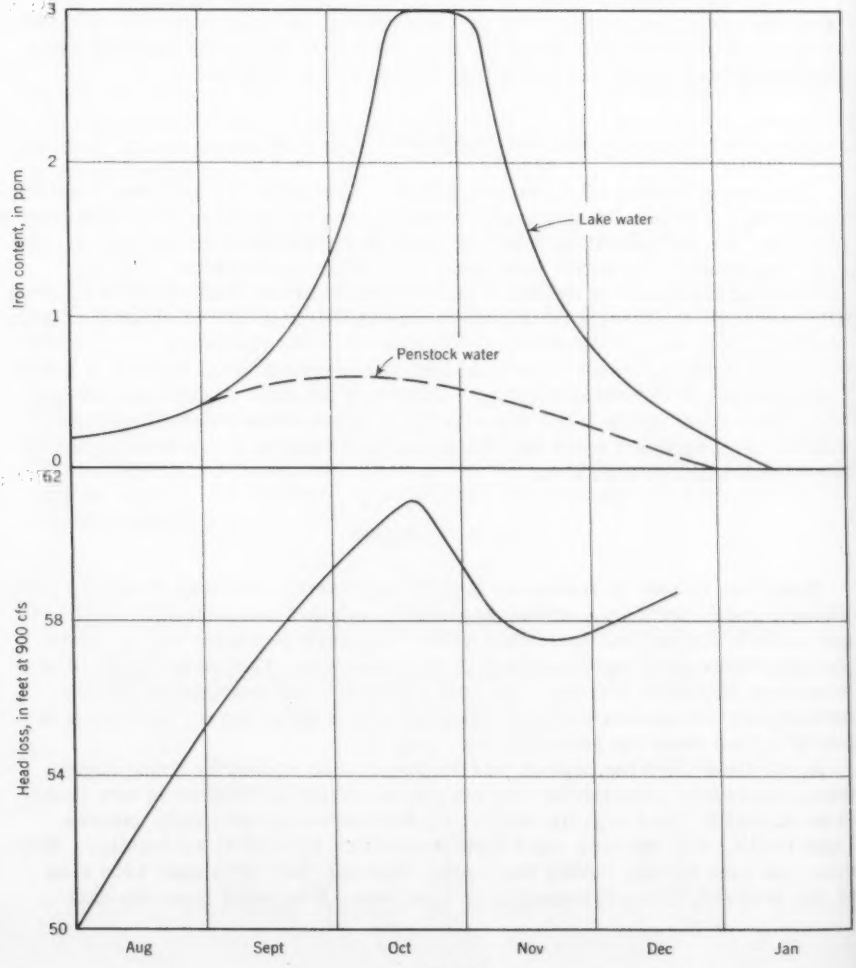

Fig. 2

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2313 December, 1959 PO 6

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POWER DIVISION

Proceedings of the American Society of Civil Engineers

SIMPLIFIED PROCEDURE FOR STRESS ANALYSIS OF GRAVITY DAMS

R. S. Sandhu[1]

ABSTRACT

By applying the formula and tables derived in this paper a quicker solution of the "Principal Stresses," by the Gravity Analysis Method, can be calculated for a gravity masonry dam. These short cuts although saving considerable man hours do not impair the accuracy of the results.

INTRODUCTION

In a masonry dam of moderate height, it is usually sufficient to compute vertical stresses on horizontal planes by Gravity Analysis. Principal stresses are calculated, ordinarily, only at the upstream and downstream faces. In a large dam, however, it is necessary to determine all the stress components at several points in the body of the structure, especially for design of reinforcement around openings.

Some of the current methods(1) employed for computing shear stresses and normal stresses on longitudinal vertical planes are rather lengthy. There is a possibility of utilizing operational short cuts, without impairing the accuracy of results, that will considerably shorten the process of obtaining principal stresses in a gravity dam. Such a possibility is explored in this paper and a simpler method of obtaining these stresses is explained. The approach suggested will, it is believed, save many precious man-hours in the design office.

To obtain a more complete picture of stress distribution in a dam, it may be desirable to include effect of load transfer to abutments through beam, arch of twist action. In a straight gravity dam, transfer of part of external load to abutments would be possible if the transverse contraction joints are either keyed or grouted. Modifications may also be made to include the effect, if any, of schedule of grouting longitudinal contraction joints in the dam. A study of these effects does not fall within the scope of this paper. Strictly speaking,

Note: Discussion open until May 1, 1959. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2313 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. PO 6, December, 1959.

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the method described herein is applicable to a cantilever(2) type gravity dam, that is, the dam is supposed to consist of a series of independently stable cantilevers, fixed at the base, and bounded by parallel transverse vertical planes.

Notation

Orthogonal planes of reference are the vertical transverse, the vertical longitudinal and the horizontal. The axes X, Y and Z are normal to the planes of reference.

- z = depth of the point considered below the top of the dam.
- y = distance of the point considered from the downstream face measured horizontally.
- T = thickness of dam at the point considered, measured horizontally.
- $k = \frac{y}{T}$
- h = depth of the point considered below the reservoir surface.
- h' = depth of the point considered below the tailwater surface.
- ϕd = inclination of downstream face of dam with vertical.
- ϕu = inclination of upstream face of dam with vertical.
- w_c = unit weight of masonry.
- w = unit weight of water.
- P_e = intensity of hydrodynamic pressure of reservoir due to horizontal earthquake (relative acceleration of dam upstream).
- P'_e = intensity of hydrodynamic pressure of tailwater due to horizontal earthquake (relative acceleration of dam downstream).
- $\lambda : 1$ = ratio in which the unit weights are affected in a vertical earthquake.
- V = horizontal force due to reservoir (positive downstream).
- V' = horizontal force due to tailwater (positive upstream).
- W = vertical force on a horizontal section (positive downwards).
- M = bending moment about center of a horizontal section (positive counterclockwise).

Assumptions

i) The dam is homogeneous and isotropic.
ii) The vertical stress, at any elevation, varies linearly from upstream to downstream face.
iii) The face slopes, in any transverse section, consist of a series of straight lines, i.e., ϕd and ϕu are constant at the point considered.

Stresses in a Dam
(See Fig. II)

Consider an elementary plane area BCD with projections ABC, ABD, ACD on planes perpendicular to X, Y and Z axes.

The traction force on the face BCD is completely defined by the nine stress components—three acting on each projection. Three of these components σ_{xx} σ_{yy} σ_{zz} are normal and the others tangential.

Under the assumption that no stress is transmitted along direction X,

$$\sigma_{xx} = \sigma_{xy} = \sigma_{xz} = 0$$

With this, the complementary tangential stresses σ_{yx} and σ_{zx} also vanish.

Thus, the only stress components to be determined are σ_{yy}, σ_{zz} and the complementary tangential stresses $\sigma_{yz} = \sigma_{zy}$

For simplicity, the duplicate suffixes may be dropped from the normal stresses and the tangential component designated as τ.

Any body forces on an element of dimensions dx, dy and dz may be represented by $\iiint \alpha w_c \, dx \, dy \, dz$, where α is a coefficient defining the intensity of the body force.

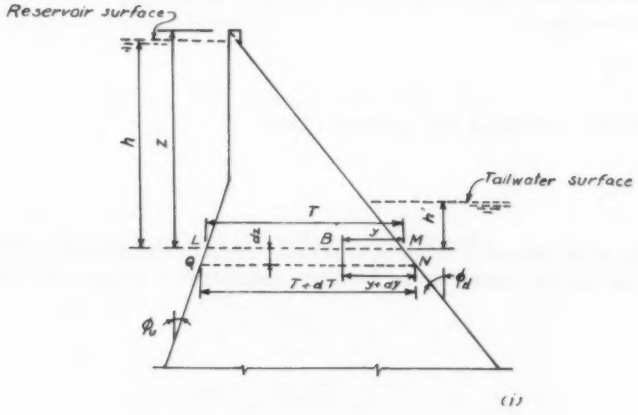

FIG I

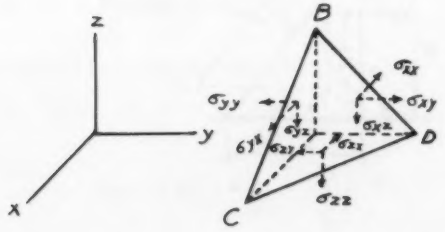

FIG. II

Hereafter, σ_z and σ_y will be treated as positive when directed so as to cause compression on the element under consideration.

Evaluation of Shear and Normal Stresses
(See Fig. III)

Let B be any point in the dam where it is desired to evaluate the stress components. Let its co-ordinates be y and z referred to the downstream edge of the horizontal plane containing it and the top of the dam, respectively.

Consider a segment ABCC' A' of the dam, bounded by two vertical transverse planes unit distance apart, horizontal and vertical longitudinal planes through B and the downstream face of dam.

By usual analysis, assuming linear variation in σ_z, σ_{zd} (suffix d denoting the downstream face).

$$= \frac{W}{T} - \frac{6M}{T^2}$$

and, σ_{zu} (suffix u denoting the upstream face)

$$= \frac{W}{T} + \frac{6M}{T^2}$$

σ_z at any point distant y from downstream edge of the horizontal plane and lying in it, assuming linear variation from downstream to upstream trace is given by

$$\sigma_{zy} = \sigma_{zd} + \frac{12M}{T^2} \cdot \frac{y}{T}$$

$$= \sigma_{zd} + (\sigma_{zu} - \sigma_{zd})\, K = K\,\sigma_{zu} + (1-K)\sigma_{zd} \qquad (1)$$

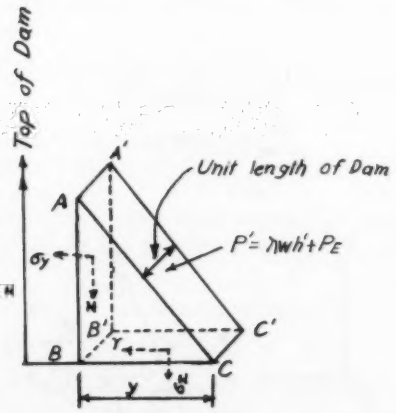

FIG. III

and also = $\frac{W}{T} + 6(2k-1)\frac{M}{T^2}$

Any body force on the segment considered would be $\iiint \alpha w_c \, dx\, dy\, dz$

$= \iint \alpha w_c \, dy\, dz$ for unit thickness of the segment.

Let the dam be subjected to a horizontal earthquake increasing the unit weights in the ratio $\lambda : 1$

Horizontal body force acting downstream

$$W_h = \iint \alpha_e w_c \, dy\, dz$$

Vertical body force acting downwards

$$W_v = \iint \lambda w_c \, dy\, dz$$

From equilibrium of vertical forces.

$$\int_{AA'}^{BB'} dz + W_v + \text{vertical component of tailwater load}$$

$$\text{on } ACC'_{A'} = \int_0^y \sigma_z \, dy$$

Differentiating with respect to z and rearranging terms,

$$= -\frac{dW_v}{dz} - (\lambda w h' + P'_e)\frac{dy}{dz} + \frac{d}{dz}\int_0^y \sigma_z \, dy$$

$$= -\lambda w_c y - (\lambda w h' + P'_e)\frac{dy}{dz} + \frac{d}{dz}\int_0^y \sigma_z \, dy$$

$$= -\lambda w_c KT - (\lambda w h' + P'_e)\tan \phi_d + \frac{d}{dz}\int_0^y \sigma_z \, dy \qquad (2)$$

From equilibrium of horizontal forces,

$$\int_{AA'}^{BB'} \sigma y \, dz + W_h = \text{Horizontal component of tailwater}$$

load on $ACC'A' + \int_0^y \tau \, dy$

Differentiating with respect to z and rearranging terms,

$$\sigma y = -\frac{dW_h}{dz} + (\lambda w h' + P'_e) + \frac{d}{dz}\int_0^y \tau \, dy$$

$$= -\alpha_e w_c y + (\lambda w h' + P'_e) + \frac{d}{dz}\int_0^y \tau \, dy$$

$$= -\alpha_e w_c y + (\lambda w h' + P'_e) - \lambda w_c y \frac{dy}{dz} - \lambda^+ w(y + h'\frac{dy}{dz})\tan \phi_d$$

$$- \left\{ P'_e \frac{dy}{dz} + y \frac{dP'_e}{dz} \right\} \tan \phi_d + \frac{d}{dz} \int_o^y \frac{d}{dz} \int_o^y \sigma_z \, dy \, dy$$

Substituting $\frac{dy}{dz} = \tan \phi_d$ and rearranging,

$$\sigma_y = -\alpha_e w_c y + (\lambda wh' + P'_e)(1 - \tan^2 \phi_d) - \lambda w_c y \tan \phi_d$$

$$- y \tan \phi_d (\lambda w + \frac{dP'_e}{dz}) + \frac{d}{dz} \int_o^y \frac{d}{dz} \int_o^y \sigma_z \, dy \, dy \qquad (3)$$

$$\int_o^y \sigma_z \, dy = \int_o^y \left[\frac{W}{T} + 6 \left(2 \frac{y}{T} - 1\right) \frac{M}{T^2} \right] dy$$

$$= \frac{W}{T} y + \frac{6M}{T^2} \left(\frac{y^2}{T} - y \right)$$

$$\frac{d}{dz} \int_o^y \sigma_z \, dy = \frac{y}{T} \frac{dW}{dz} + \frac{W}{T} \frac{dy}{dz} - \frac{yW}{T^2} \frac{dT}{dz} + 6 \left[\frac{y^2}{T} - y \right] \frac{1}{T^2} \frac{dM}{dz}$$

$$+ \frac{6M}{T^2} \left(\frac{2y}{T} \frac{dy}{dz} - \frac{dy}{dz} - \frac{y^2}{T^2} \frac{dT}{dz} \right) - \frac{2}{T} \left(\frac{y^2}{T} - y \right) \frac{dT}{dz}$$

Now, $\frac{dy}{dz} = \tan \phi_d$

$$\frac{dT}{dz} = \tan \phi_d + \tan \phi_u$$

$$\frac{y}{T} = k$$

$$\frac{dV}{dz} = (\lambda wh + P_e) - (\lambda wh' + P'_e) + \alpha_e w_c T$$

$$\frac{dW}{dz} = \lambda w_c T + (\lambda wh + P_e) \tan \phi_u + (\lambda wh' + P'_e) \tan \phi_d$$

$$\frac{dM}{dz} = \frac{W}{2} (\tan \phi_d - \tan \phi_u) - V + \frac{T}{2} (\lambda wh + P_e) \tan \phi_u$$

$$- \frac{T}{2} (\lambda wh' + P'_e) \tan \phi_d$$

Therefore, $\frac{d}{dz} \int_o^y \sigma_z \, dy = \lambda w_c kT + k(\lambda wh + P_e) \tan \phi_u + k(\lambda wh' + P'_e) \tan \phi_d$

$$+ \frac{M}{T} \left[(1-k) \tan \phi_d - k \tan \phi_u \right]$$

$$+ 6k(k-1) \left[\frac{W}{2T} (\tan \phi_d - \tan \phi_u) - \frac{V}{T} + \frac{1}{2} (\lambda wh + P_e) \tan \phi_u \right.$$

---

* This term vanishes with no tailwater.

$$- \tfrac{1}{2}(\lambda w h' + P'_e)\tan\phi_d\Big]$$

$$+ \frac{6M}{T^2}\Big[(2k=1)\tan\phi_d - k^2(\tan\phi_d + \tan\phi_u)$$

$$- 2k(k-1)(\tan\phi_d + \tan\phi_u)\Big]$$

$$= \lambda w_c kT + (\lambda wh + P_e)k(3k-2)\tan\phi_u$$

$$+ (\lambda wh' + P'_e)\,k\,(4-3k)\tan\phi_d$$

$$+ \frac{V}{T}\,6k(1-k) + \frac{W}{T}\Big[(1-k)\,1-3k)\tan\phi_d$$

$$- k(3k-2)\tan\phi_u\Big]$$

$$+ \frac{M}{T^2}\,6\Big[-(1-k)(1-3k)\tan\phi_d - k(3k-2)\tan\phi_u\Big] \qquad (4)$$

$$\int_o^y \frac{d}{dz}\int_o^y \sigma_z\,dy\,dy = \lambda w_c\,\frac{y^2}{2} + \left(\frac{2y^2}{T} - \frac{y^3}{T^2}\right)(\lambda wh' + P')\tan\phi_d$$

$$+ \left(\frac{y^3}{T^2} - \frac{y^2}{T}\right)(\lambda wh + P)\tan\phi_u + \left(\frac{3y^2}{T} - \frac{2y^3}{T^2}\right)\frac{V}{T}$$

$$+ \left[\left\{y - \frac{2y^2}{T} + \frac{y^3}{T^2}\right\}\tan\phi_d - \left(\frac{y^3}{T^2} - \frac{y^2}{T}\right)\tan\phi_u\right]\frac{W}{T}$$

$$+ \left[-\left\{y - \frac{2y^2}{T} + \frac{y^3}{T^2}\right\}\tan\phi_d - \left(\frac{y^3}{T^2} - \frac{y^2}{T}\right)\tan\phi_u\right]\frac{6M}{T^2}$$

$$\text{and } \frac{d}{dz}\int_o^y \frac{d}{dz}\int_o^y \sigma_z\,dy\,dy = \lambda w_c\,y\tan\phi_d + \Big[k(4-3k)\tan\phi_d$$

$$- 2k^2(1-k)\tan\phi_d - 2k^2(1-k)\tan\phi_u\Big]\tan\phi_d\,(\lambda wh' + P'_e)$$

$$+ \Big[k(3k-2)\tan\phi_d$$

$$- k^2(2k-1)\tan\phi_d - k^2(2k-1)\tan\phi_u\Big]\tan\phi_u\,(\lambda wh + P_e)$$

$$+ k^2(2-k)\,T\tan\phi_d\left[\lambda w^+ + \frac{dP'_e}{dz}\right]$$

$$+ k^2(k-1)T\tan\phi_u\left[\lambda w + \frac{dP_e}{dz}\right]$$

$$+ 6k(1-k)\tan\phi_d\,\frac{V}{T} + 6k^2(k-1)(\tan\phi_d + \tan\phi_u)\,\frac{V}{T}$$

$$+ k^2(3-2k)\left[(\lambda wh + P_e) - (\lambda wh' + P'_e) + \alpha_e w_c T\right]$$

$$+ \Big[\Big\{(1-4k+3k^2)\tan\phi_d + k(2-3k)\tan\phi_u\Big\}\tan\phi_d$$

$$+\left\{-k(1-4k+3k^2)\tan\phi_d + k^2(-2+3k)\tan\phi_u\right\}(\tan\phi_d+\tan\phi_u)\Big]\frac{W}{T}$$

$$+\left\{k(1-k)^2\tan\phi_d + k^2(1-k)\tan\phi_u\right\}\Big[\lambda w_c T + (\lambda wh' + P'_e)\tan\phi_d + (\lambda wh + P_e)\tan\phi_u\Big]$$

$$+\Big[\left\{-(1-4k+3k^2)\tan\phi_d + k^2(2-3k)\tan\phi_u\right\}\tan\phi_d + \left\{(2k-6k^2+4k^3)\tan\phi_d - k^2(3-4k)\tan\phi_u\right\}(\tan\phi_d+\tan\phi_u)\Big]\frac{6M}{T^2}$$

$$+\Big[-(k-2k^2+k^3)\tan\phi_d + k^2(1-k)\tan\phi_u\Big]\Big[3(\tan\phi_d - \tan\phi_u)\frac{W}{T} - \frac{6V}{T}$$

$$-3(\lambda wh' + P'_e)\tan\phi_d + 3(\lambda wh + P_e)\tan\phi_u$$

$$= \alpha_e w_e T k^2 (3-2k)$$

$$+\lambda w_c Tk\Big[\tan\phi_d + (1-k)\left\{(1-k)\tan\phi_d + k\tan\phi_u\right\}\Big]$$

$$+\lambda w\, Tk\Big[\left\{k(2-k)\tan\phi_d\right\}^* + k(k-1)\tan\phi_u\Big]$$

$$+(\lambda wh' + P_e)\Big[K^2(3-2k) + \left\{k^2(5-6k)\tan\phi_u - 4k(1-k)^2\tan\phi_d\right\}\tan\phi_u\Big]$$

$$+(\lambda wh' + P'_e)\Big[-k^2(3-2k) + \left\{-4k^2(1-k)\tan\phi_u + k(8-13k+6k^2)\tan\phi_d\right\}\tan\phi_d\Big]$$

$$+T\frac{dP_e}{dz}k^2(k-1)\tan\phi_u + T\frac{dP'_e}{dz}k^2(2-k)\tan\phi_d$$

$$+\frac{V}{T}12k(1-k)\Big[(1-k)\tan\phi_d - k\tan\phi_u\Big]$$

$$+\frac{W}{T}\Big[(1-k)^2(1-6k)\tan^2\phi_d + 4k(1-k)\tan\phi_d\tan\phi_u + k^2(-5+6k)\tan^2\phi_u\Big]$$

$$+\frac{6M}{T^2}\Big[-(1-k)^2(1-4k)\tan^2\phi_d + 4k(1-k)(1-2k)\tan\phi_d\tan\phi_u + k^2(-3+4k)\tan^2\phi_u\Big] \qquad (5)$$

Thus, from equations (2) and (4)

$$\tau = -\lambda w_c KT - (\lambda wh' + P'_e) \tan \phi_d + \frac{d}{dz} \int_o^y \sigma_z \, dy$$

$$= -(\lambda wh' + P'_e)(1-k)(1-3k) \tan \phi_d + (\lambda wh + P_e)\, k(3k-2) \tan \phi_u$$

$$+ \frac{V}{T} 6k(1-k) + \frac{W}{T} \left[(1-k)(1-3k) \tan \phi_d - k(3k-2) \tan \phi_u\right]$$

$$+ \frac{6M}{T^2} \left[-(1-k)1-3k) \tan \phi_d - k(3k-2) \tan \phi_u\right]$$

From equilibrium of vertical forces on infinitesimal triangular prisms at upstream and downstream faces,

$$\tau_u = (\lambda wh + P_e) \tan \phi_u - \sigma_{zu} \tan \phi_u$$

$$= (\lambda wh + P_e) \tan \phi_u - \left\{\frac{W}{T} + \frac{6M}{T^2}\right\} \tan \phi_u$$

and

$$\tau_d = \sigma_{zd} \tan \phi_d - (\lambda wh' + P'_e) \tan \phi_d$$

$$= -(\lambda wh' + P'_e) \tan \phi_d + \left\{\frac{W}{T} - \frac{6M}{T^2}\right\} \tan \phi_d$$

Substituting these values in the expression for τ, we have

$$\tau = C' \tau_d + C'' \tau_u + C''' \frac{V}{T} \qquad (6)$$

Where $C' = (1-k)(1-3k)$
$C'' = k(3k-2)$
$C''' = 6k(1-k)$

These coefficients are independent of the dam slopes and therefore, values in Table I have universal application. Other interesting relations between these coefficients are:

$$C' + C'' + C''' = 1$$

and

$$C'_{(k)} = C''_{(1-k)}$$

Again from equations (3) and (5)

$$\sigma_y = -\alpha_e w_c kT + (\lambda wh' + P'_e)(1 - \tan^2 \phi_d) - \lambda w_c y \tan \phi_d$$

$$- y \tan \phi_d \left(\lambda w^* \frac{dP'_e}{dz}\right) + \frac{d}{dz} \int_o^y \frac{d}{dz} \int_o^y \sigma_z \, dy \, dy$$

$$= \alpha_e w_c Tk(-1+3k-2k^2)$$

$$+ \lambda w_c Tk(1-k) \left[(1-k) \tan \phi_d + k \tan \phi_u\right]$$

$$+ \lambda w \, Tk \left[\left\{-(1-k)^2 \tan \phi_d\right\}^* + k(k-1) \tan \phi_u\right]$$

$$+ (\lambda wh + P_e) \left[k^2(3-2k) + \left\{k^2(5-6k) \tan \phi_u - 4k(1-k)^2 \tan \phi_d\right\} \tan \phi_u\right]$$

$$+ (\lambda wh' + P'_e) \left[1 - k^2(3-2k) + \left\{-4k^2(1-k) \tan \phi_u\right.\right.$$

(5)

$$-(1-k^2\ (1-6k)\ \tan\phi_d\Big\}\tan\phi_d\Big]$$

$$-T\ \frac{dP_e}{dz}\ k^2(1-k)\tan\phi_u\ -T\ \frac{dP'_e}{dz}\ k(1-k)^2\ \tan\phi_d$$

$$+\frac{V}{T}\ 12k\ (1-k)\left[(1-k)\ \tan\phi_d\ -k\ \tan\phi_u\right]$$

$$+\frac{W}{T}\left[(1-k)^2(1-6k)\ \tan^2\phi_d+4k(1-k)\ \tan\phi_d\ \tan\phi_u\right.$$

$$\left.+k^2(-5+6k)\ \tan^2\phi_u\right]$$

$$+\frac{6M}{T^2}\left[-(1-k)^2\ (1-4k)\ \tan^2\phi_d+4k(1-k)\ (1-2k)\ \tan\phi_d\ \tan\phi_u\right.$$

$$\left.+k^2(-3+4k)\ \tan^2\phi_u\right]$$

By substituting values of $\mathcal{T}_d$ and $\mathcal{T}_u$,

$$\sigma y = -\ \frac{\alpha}{e} w_c Tk(1-k)\ (1-2k)+\lambda\ w_c Tk(1-k)\left[(1-k)\ \tan\phi_d+k\ \tan\phi_u\right]$$

$$-\lambda w\ Tk(1-k)\left[\left\{(1-k)\ \tan\phi_d\right\}^*+k\ \tan\phi_u\right]$$

$$+(\lambda wh+P_e)k^2\ (3-2k)+(\lambda wh'+P'_e)\ (1-k)^2\ (1+2k)$$

$$+\mathcal{T}_d\left[(1-k)^2\ (1-6k)\ \tan\phi_d+4k^2\ (1-k)\ \tan\phi_u\right]$$

$$+\mathcal{T}_u\left[-k^2(-5+6k)\ \tan\phi_u\ -4k\ (1-k)^2\ \tan\phi_d\right]$$

$$+\frac{V}{T}\ 12k(1-k)\left[(1-k)\ \tan\phi_d\ -k\ \tan\phi_u\right]$$

$$+\frac{M}{T^2}\ 12k(1-k)\left[-(1-k)\ \tan^2\phi_d+k\ \tan^2\phi_u\right]$$

$$-T\ \frac{dP_e}{dz}\ k^2\ (1-k)\ \tan\phi_u\ -T\ \frac{dP'_e}{dz}\ k(1-k)^2\ \tan\phi_d$$

$$=C_1P\ +\ C_2\ P'+C_3\ T+C_4\ \mathcal{T}_d+C_5\ \mathcal{T}_u+C_5\ \frac{V}{T}\ +C_7\ \frac{M}{T^2}$$

$$+C_8\ T\ \frac{dP_e}{dz}+C_9\ T\ \frac{dP'_e}{dz} \qquad (6)$$

Where P = Intensity of reservoir pressure on upstream face
P' = Intensity of tailwater pressure on downstream face
$C_1 = k^2(3-2k)$

$$C_2 = (1-k)^2 (1+2k) = C_1 (1-K) = 1-C_1$$

$$C_3 = k(1-k) \left[-\alpha_e w_c (1-2k) + \lambda w_c (1-k) \tan \phi_d + \lambda w_c k \tan \phi_u \right.$$
$$\left. -\{\lambda w (1-k) \tan \phi_d\}^* - \lambda w k \tan \phi_u \right]$$
$$- \alpha_e w_c k_1 + \lambda w_c (k_2 \tan \phi_u + k_3 \tan \phi_d)$$
$$- \lambda w \left[(k_2 \tan \phi_u) + (k_3 \tan \phi_d)^* \right]$$

$$C_4 = 4k_2 \tan \phi_u + k_4 \tan \phi_d$$

$$C_5 = -k_5 \tan \phi_u - 4k_3 \tan \phi_d$$

$$C_6 = -12k_2 \tan \phi_u + 12k_3 \tan \phi_d = 12(C_8 - C_9)$$

$$C_7 = 12k_2 \tan^2 \phi_u - 12k_3 \tan^2 \phi_d$$

$$C_8 = -k_2 \tan \phi_u$$

$$C_9 = -k_3 \tan \phi_d$$

and

$$k_1 = k(1-k)(1-2k)$$

$$k_2 = k^2 (1-k)$$

$$k_3 = k(1-k)^2 = k_2 (1-k)$$

$$k_4 = (1-k)^2 (1-6k)$$

$$k_5 = k^2 (-5+6k) = k_4 (1-k)$$

Term marked * is to be ignored when there is no tail water.

The only coefficient depending upon external and body forces is C_3 which can be tabulated for various combinations, desired to be considered, of these forces.

For no earthquake and no tailwater, the expression for σ_y reduces to,

$$\sigma_y = C_1 wh + C_3 T + C_4 \gamma_d + C_5 \gamma_u + C_6 \frac{V}{T} + C_7 \frac{M}{T^2}$$

Values of C_1, C_2 and k_1 through k_5 for various values of k are included in Table I.

Summary of Results

Equations for normal and shear stresses in a Gravity Dam are:

$$\sigma_z = k \sigma_{zu} + (1-k) \sigma_{zd}$$

$$\mathcal{T} = C' \mathcal{T}_d + C'' \mathcal{T}_u + C''' \frac{V}{T}$$

$$\sigma_y = C_1 P + C_2 P' + C_3 T + C_4 \mathcal{T}_d + C_5 \mathcal{T}_u + C_6 \frac{V}{T} + C_7 \frac{M}{T^2}$$

$$+ C\ T \frac{dP_e}{dz} + C\ T \frac{dP'_e}{dz}$$

where

$C' = (1-k)(1-3k)$

$C'' = k(3k-2)$

$C''' = 6k(1-k)$

$C_1 = k^2 (3-2k)$

$C_2 = (1-k)^2 (1+2k)$

$C_3 = -\alpha_e w_c k_1 + \lambda w_c (k_2 \tan \phi_u + k_3 \tan \phi_d)$

$- \lambda w \left[k_2 \tan \phi_u + (k_3 \tan \phi_d) \right]^*$

TABLE OF UNIVERSAL CO-EFFICIENTS Table 1

K	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
C'	1.00	0.63	0.32	0.07	-0.12	-0.25	-0.32	-0.33	-0.28	-0.17	0.00
C''	-0.00	-0.17	-0.28	-0.33	-0.32	-0.25	-0.12	0.07	0.32	0.63	1.00
C'''	0.00	0.54	0.96	1.26	1.44	1.50	1.44	1.26	0.96	0.54	0.00
C1	0.000	0.028	0.104	0.216	0.352	0.500	0.648	0.784	0.896	0.972	1.000
C2	1.000	0.972	0.896	0.784	0.648	0.500	0.352	0.216	0.104	0.028	0.000
K1	0.000	0.072	0.096	0.084	0.048	0.000	-0.048	-0.084	-0.096	-0.072	0.000
K2	0.000	0.009	0.032	0.063	0.096	0.125	0.144	0.147	0.128	0.081	0.000
K3	0.000	0.081	0.128	0.147	0.144	0.125	0.096	0.063	0.032	0.009	0.000
K4	1.000	0.324	-0.128	-0.392	-0.504	-0.500	-0.416	-0.288	-0.152	-0.044	0.000
K5	0.000	-0.044	-0.152	-0.288	-0.416	-0.500	-0.504	-0.392	-0.128	0.324	1.000

$$C_4 = 4k_2 \tan \phi + k_4 \tan \phi_d$$

$$C_5 = -k_5 \tan \phi_u - 4k_3 \tan \phi_d$$

$$C_6 = -12k_2 \tan \phi_u + 12k_3 \tan \phi_d$$

$$C_7 = 12k_2 \tan^2 \phi_u - 12k_3 \tan^2 \phi_d$$

$$C_8 = -k_2 \tan \phi_u$$

$$C_9 = -k_3 \tan \phi_d$$

$$k_1 = k(1-k)(1-2k)$$

$$k_2 = k^2(1-k)$$

$$k_3 = k(1-k)^2$$

$$k_4 = (1-k)^2(1-6k)$$

$$k_5 = k^2(-5+6k)$$

Term with superscript * to be ignored if there is no tailwater.

Procedure for Gravity - Analysis

Coefficients C', C'', C''', C_1, C_2 and k's, the auxiliary coefficients, are independent of dam slopes, and tables for these given in this paper hold in all cases. C_3 through C_9 can be worked out and plotted for the case under consideration from k's, as shown in Tables II and III for a dam having downstream slope 0.8 to 1 and upstream face vertical for top 350 feet and sloping at 0.35 to 1 below that. Values of Z, T, W, M,

$$\frac{W}{T}, \frac{M}{T^2}, P, P', V, \frac{V}{T}, \frac{6M}{T^2}, \sigma_{zd}, \sigma_{zu}, \sigma_{zd}-P', p-\sigma_{zu}, \gamma_d, \gamma_u, \frac{dP_e}{dz}, \frac{dP'_e}{dz}$$

may be tabulated in the usual manner for various desired elevations under the loading conditions considered. For complete determination of stress at any point all the work required now consists in reading off values from Tables (or curves may be plotted), multiplying the corresponding terms and adding up.

REFERENCES

1. "Treatise on Dams." Chapter 9, Gravity Dams Design Supplement No. 2, Vol. X.
Reclamation Manual. U. S. Bureau of Reclamation 1950.

2. "Monolithic and Non-monolithic Gravity Dams," by G. S. Sarkaria, Water Power. London 1955.

GRAVITY ANALYSIS _____ Table II

EVALUATION OF DERIVED CO-EFFICIENTS :- C_3 thru' C_9

K	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
$-k_3 \tan \beta_d$	0	-0.064,8	-0.102,4	-0.117,6	-0.115,2	-0.100,0	-0.076,8	-0.050,4	-0.025,6	-0.007,2	0
$k_2 \tan \beta_u$	0	0.003,15	0.011,20	0.022,05	0.033,60	0.043,75	0.050,40	0.051,45	0.044,80	0.028,35	0
$4k_2 \tan \beta_u$	0	0.012,6	0.044,8	0.088,2	0.134,4	0.175,0	0.201,6	0.205,8	0.179,2	0.113,4	0
$k_4 \tan \beta_d$	0.800,0	0.259,2	-0.102,40	-0.313,6	-0.403,2	-0.400,0	-0.332,8	-0.230,4	-0.121,6	-0.035,2	0
for Z^{C_4} 350	0.800,0	0.271,8	-0.057,6	-0.225,4	-0.288,8	-0.225,0	-0.131,2	-0.024,6	0.057,6	0.078,2	0
$-k_5 \tan \beta_u$	0	0.015,4	0.053,2	0.100,8	0.145,6	0.175,0	0.176,4	0.137,2	0.044,8	-0.113,4	-0.350,0
$-4k_3 \tan \beta_d$	0	-0.259,2	-0.409,6	-0.470,4	-0.460,8	-0.400,0	-0.307,2	-0.201,6	-0.102,4	-0.028,8	0
for Z^{C_5} 350	0	-0.243,8	-0.356,4	-0.369,6	-0.315,2	-0.225,0	-0.130,8	-0.064,4	-0.057,6	-0.142,2	-0.350,0
$-12k_2 \tan \beta_d$	0	-0.037,8	-0.134,4	-0.264,6	-0.403,2	-0.525,0	-0.604,8	-0.617,4	-0.537,6	-0.340,2	0
$12k_3 \tan \beta_d$	0	0.777,6	1.285,8	1.411,2	1.382,4	1.200,0	0.921,6	0.604,8	0.307,2	0.086,4	0
for Z^{C_6} 350	0	0.739,8	1.094,4	1.146,6	0.979,2	0.675,0	0.316,8	-0.012,6	-0.230,4	-0.253,8	0
$12k_2 \tan^2 \beta_u$	0	0.013,23	0.047,04	0.092,61	0.141,12	0.183,75	0.211,68	0.216,09	0.188,16	0.118,91	0
$12k_3 \tan^2 \beta_d$	0	-0.622,08	-0.983,04	-1.128,96	-1.105,92	-0.960,00	-0.737,28	-0.493,84	-0.245,76	-0.069,12	0
for Z^{C_7} 350	0	-0.608,85	-0.936,00	-1.036,35	-0.964,80	-0.776,25	-0.525,60	-0.267,75	-0.057,60	0.049,95	0
$v_o k_1 \tan \beta_d$	0	0.009,979	0.018,770	0.018,110	0.017,741	0.015,400	0.011,827	0.007,762	0.003,942	0.001,109	0
$v_o k_2 \tan \beta_u$	0	0.000,485	0.001,725	0.003,396	0.005,174	0.006,738	0.007,762	0.007,923	0.006,899	0.004,366	0
$-v_o k_2 \tan \beta_u$	0	-0.000,197	-0.000,700	-0.001,378	-0.002,100	-0.002,734	-0.003,150	-0.003,216	-0.002,800	-0.001,772	0
$-4k_3 \tan \beta_d$	0	-0.004,050	-0.006,400	-0.007,350	-0.007,200	-0.006,250	-0.004,800	-0.003,150	-0.001,600	-0.000,450	0
$-v_o v_o k_1$	0	0.001,663	0.002,816	0.001,940	0.001,109	0	-0.001,109	-0.001,940	-0.002,816	-0.001,663	0

GRAVITY ANALYSIS

Table III

TABLE OF DERIVED CO-EFFICIENTS :- C_3 thru' C_9 .

K	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
C_3^* 37.5<Z<350	0	0.009,979	0.015,770	0.018,110	0.017,741	0.015,400	0.011,827	0.007,761	0.003,942	0.001,109	0
C_3^* Z>350 Dam weight only	0	0.010,464	0.017,494	0.021,506	0.022,915	0.022,136	0.019,569	0.015,685	0.010,842	0.005,475	0
C_3^* Z>350 Dam wt + Reservoir	0	0.010,207	0.016,794	0.020,128	0.020,615	0.019,403	0.016,439	0.012,468	0.008,042	0.003,703	0
C_3^* Z>350 Dam wt + Reservoir wind +	0	0.006,217	0.010,394	0.012,778	0.013,615	0.013,153	0.011,639	0.009,319	0.006,442	0.003,253	0
C_3^* Additional for Horiz. Earqke	0	-0.001,663	-0.002,218	-0.001,940	-0.001,109	0	0.001,109	0.001,940	0.002,218	0.001,663	0
For Z<350											
C_4	0.800,0	0.259,2	-0.102,4	-0.313,6	-0.403,2	-0.400,0	-0.332,8	-0.230,4	-0.121,6	-0.035,2	0
C_5	0	-0.259,2	-0.409,6	-0.470,4	-0.460,8	-0.400,0	-0.307,2	-0.201,6	-0.102,4	-0.028,8	0
C_6	0	0.777,6	1.228,8	1.411,2	1.382,4	1.200,0	0.921,6	0.604,8	0.307,2	0.086,4	0
C_7	0	-0.622,08	-0.983,04	-1.128,96	-1.105,92	-0.960,00	-0.737,28	-0.483,84	-0.245,76	-0.069,12	0
C_8	←				NIL						→
C_9	0	-0.064,8	-0.102,4	-0.117,6	-0.115,2	-0.100,0	-0.076,8	-0.050,4	-0.025,6	-0.007,2	0
For Z>350											
C_4	0.800,0	0.271,8	-0.057,6	-0.225,4	-0.268,8	-0.225,0	-0.131,2	-0.024,6	0.057,6	0.078,2	0
C_5	0	-0.243,8	-0.356,4	-0.369,6	-0.315,2	-0.225,0	-0.130,8	-0.064,4	-0.057,6	-0.142,2	-0.350,0
C_6	0	0.739,8	1.094,4	1.146,6	0.979,2	0.675,0	0.316,8	-0.012,6	-0.230,4	-0.253,8	0
C_7	0	-0.608,85	-0.936,00	-1.036,35	-0.964,80	-0.776,25	-0.525,60	-0.267,75	-0.057,60	0.049,95	0
C_8	0	-0.003,15	-0.011,20	-0.022,05	-0.033,60	-0.043,75	-0.050,40	-0.051,45	-0.044,80	-0.028,35	0
C_9	0	-0.064,8	-0.102,4	-0.117,6	-0.115,2	-0.100,0	-0.076,8	-0.050,4	-0.025,6	-0.007,2	0

* In case of vertical earthquake, multiply the co-efficient by λ

† For all values of Z.

†α_e assumed to be equal to 0.15g

Journal of the

POWER DIVISION

Proceedings of the American Society of Civil Engineers

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Note: Paper 2328 is part of the copyrighted Journal of the Power Division, Proceedings of the American Society of Civil Engineers, Vol. 85, PO 6, December, 1959.

ROCKFILL DAMS: BROWNLEE SLOPING CORE DAM[a]

Discussion by G. H. Condit

G. H. CONDIT.[1]—An attempt has been made to follow the movement of Brownlee dam as closely as possible. It is unfortunate that construction methods do not make it possible to begin measurements soon enough to plot initial settlement and horizontal movement. Monuments for measurement purpose were not placed along the crest until all cable trench work and final dressing along the top was completed, so the possibility of damage to the monuments would be greatly reduced.

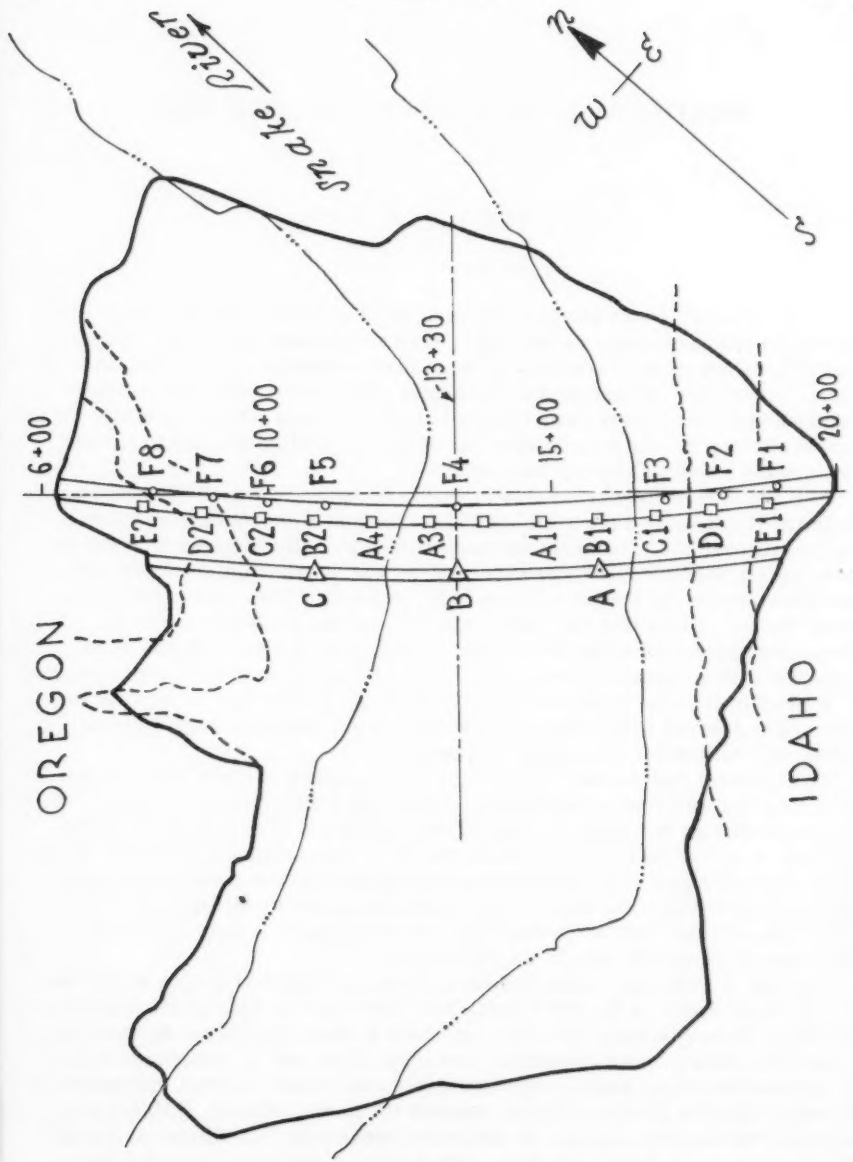
The position of the monuments installed is shown in Fig. 1. Monuments A, B and C are placed along an upstream bench at Elev. 2050; these monuments are used for settlement measurement only. Readings were begun on these points four months before those along the top and continued for three months after the others were begun which permits a correlation with those along the top. Since that time they have been under the water surface. Monuments along the crest of the dam consist of a minimum of one cy of concrete with an imbedded brass cap. The "F" settlement rods are supported on a bearing plate on the top of the clay core. They are set inside a 16 inch pipe and supported at intervals by a spider to provide an accurate reading for settlement but not for horizontal movement.

Total lateral and vertical displacement for monuments and vertical displacement for settlement rods on September 15, 1959 is shown on Fig. 2. It will be noted that the maximum displacement for the two types of readings does not occur at the same station on the dam. Maximum total lateral movement measured was at A4 which is in area of deepest toe trench excavation and at an area where the finer river sands and gravels existed in the river bed. The greatest vertical settlement has taken place at Station 13 + 13 which was the deepest area in the core trench.

For Fig. 3 settlement readings for approximate quarter points on the dam and the monuments on the 2050 bench have been plotted against the headwater elevation. It can be seen that there has been a distinct relation between the settlement rates and the headwater elevation. The rate of settlement lagged the headwater change rate by approximately one month. Lateral movement of A4, which had the greatest change, showed the same relation. Table 1 is a listing of the monthly changes in horizontal movement. In all cases, except one, there was a definite upstream shift following the lowering of the water. Table 2 is a similar tabulation for settlement.

No axial movement readings were taken until September 2, 1959. As would be expected there is a definite movement axially from each end of the dam

a. Proc. Paper 1734, August, 1958, by Torald Mundal.
1. Resident Engr., Idaho Power Company.

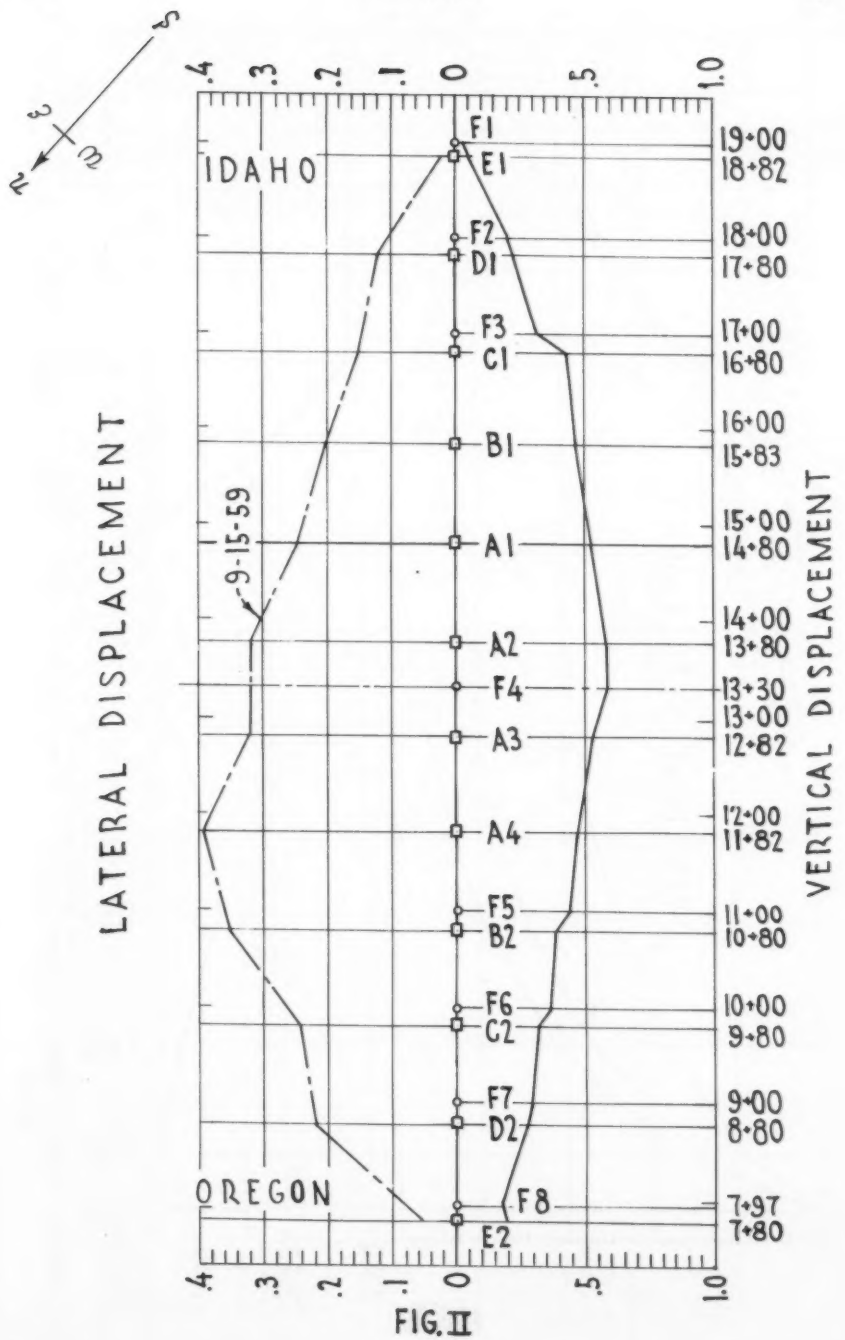


MONUMENT LOCATION PLAN

1 0 1 2 3 4 5

Scale In Hundreds Of Feet

FIG. I


S
E
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.2
.1
0
.5
1.0
IDAHO
OREGON
F1
E1
F2
D1
F3
C1
B1
A1
A2
F4
A3
A4
F5
B2
F6
C2
F7
D2
F8
E2
9-15-59
19+00
18+82
18+00
17+80
17+00
16+80
16+00
15+83
15+00
14+80
14+00
13+80
13+30
13+00
12+82
12+00
11+82
11+00
10+80
10+00
9+80
9+00
8+80
7+97
7+80
LATERAL DISPLACEMENT
VERTICAL DISPLACEMENT

FIG. II

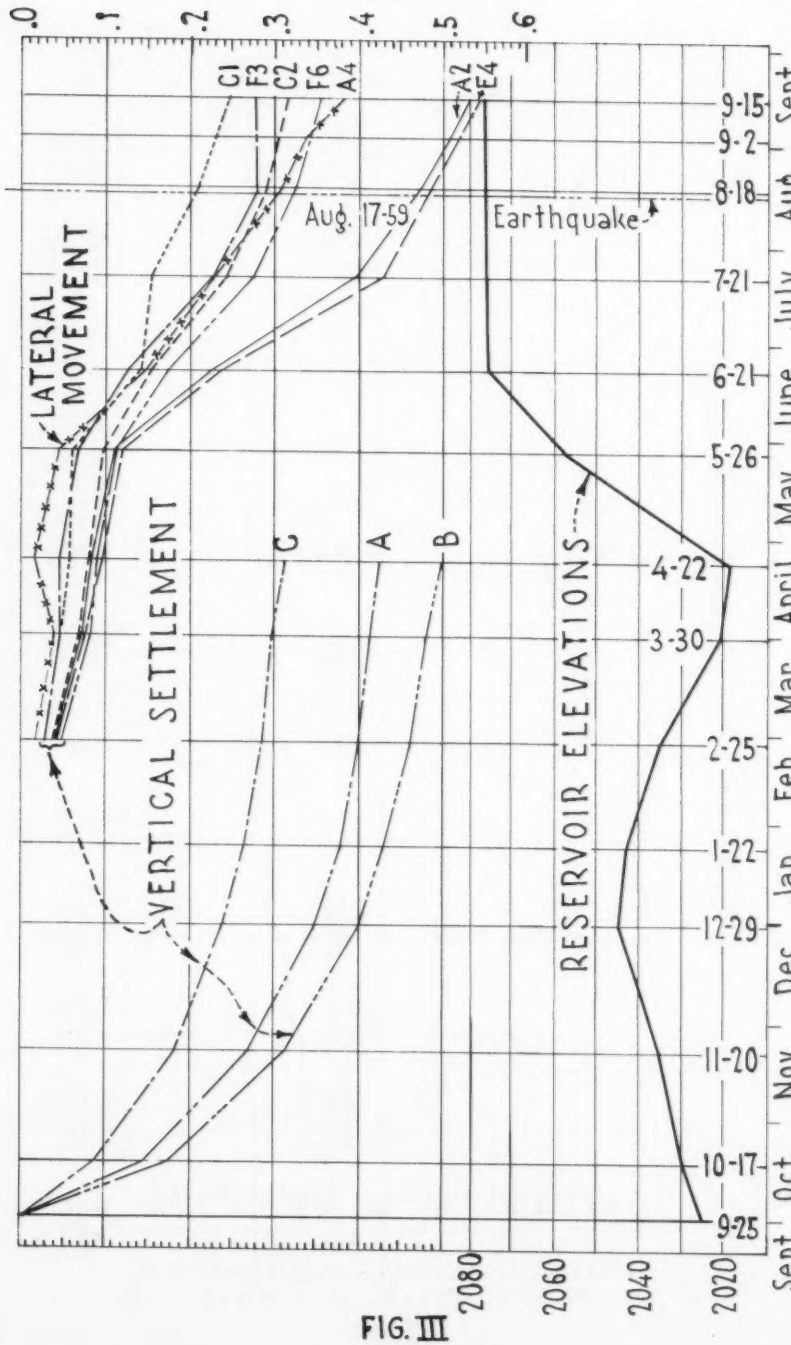
LATERAL MOVEMENT
VERTICAL SETTLEMENT
RESERVOIR ELEVATIONS
C1
F3
C2
F6
A4
A2
E4
C
A
B
Aug. 17-59
Earthquake
.0
.1
.2
.3
.4
.5
.6
2080
2060
2040
2020
9-15
9-2
8-18
7-21
6-21
5-26
4-22
3-30
2-25
1-22
12-29
11-20
10-17
9-25
Sept.
Aug.
July
June
May
April
Mar.
Feb.
Jan.
Dec.
Nov.
Oct.
Sept.
1958
1959

FIG. III

1958 1959

TABLE 1

DAM HORIZONTAL MOVEMENT SINCE PREVIOUS MONTH

Mon No	Oct 17 1958	Nov 20 1958	Dec 29 1958	Jan 22 1959	Feb 25 1959	Mar 30 1959	Apr 22 1959	May 26 1959	June 21 1959	July 21 1959	Aug 18 1959	Sept 2 1959	Sept 15 1959	Total
HW	2029.0	2034.9	2044.6	2043.7	2034.7	2020.3	2018.2	2057.0	2076.1	2076.2	2076.5	2076.5	2076.3	26,664.0
A	.14	.13	.09	-.03	-.005	.005	-.05							.28
B	.21	.05	.09	.02	.04	.02	-.07							.36
C	.13	.04	.07	.025	.00	.005	-.06							.21
E-2-X												.00		.00
E-2-Y					.02	-.02	-.02	.03	.00	.04	-.01	.01	.00	.05
D-2-X												.12		.12
D-2-Y					.04	-.01	-.01	.05	.04	.06	.05	.00	.01	.23
C-2-X												.06		.06
C-2-Y					.00	.00	.00	.03	.04	.12	.04	.01	.00	.24
B-2-X												.17		.17
B-2-Y					.03	.03	-.04	.02	.13	.12	.04	.01	.01	.35
A-4-X												.06		.06
A-4-Y					.02	.02	-.02	.03	.11	.08	.08	.03	.05	.40
A-3-X												.03		.03
A-3-Y					.00	.03	-.04	.02	.10	.09	.10	.00	.01	.31
A-2-X												-.06		-.06
A-2-Y					.02	.01	-.02	.02	.06	.15	.03	.02	.05	.34
A-1-X												-.09		-.09
A-1-Y					.00	-.02	+.02	-.02	.11	.06	.04	.05	.02	.26
B-1-X												-.11		-.11
B-1-Y					.00	.01	-.05	.04	.04	.09	.08	.00	.00	.21
C-1-X												-.17	.	-.17
C-1-Y					.00	.01	-.01	.00	.06	.09	.00	.00	.00	.15
D-1-X												-.29		-.29
D-1-Y					.00	.03	-.03	.00	.04	.04	.01	.00	.03	.12
E-1-X												-.15		-.15
E-1-Y					.00	.00	.00	.00	.04	-.04	-.02	.03	.01	.02

TABLE 2

DAM SETTLEMENT SINCE PREVIOUS MONTH

Mon No	Sept 25 1958	Oct 17 1958	Nov 20 1958	Dec 29 1958	Jan 22 1959	Feb 25 1959	Mar 30 1959	Apr 22 1959	May 26 1959	June 21 1959	July 21 1959	Aug 18 1959	Sept 15 1959	Total
HW	2023.9	2029.0	2034.9	2044.6	2043.7	2034.7	2020.3	2018.2	2057.0	2076.1	2076.2	2076.5	2076.3	26,611.4
A	0	.148	.123	.080	.028	.022	.013	.009						.423
B	0	.173	.140	.087	.033	.028	.019	.017						.497
C	0	.092	.089	.060	.023	.021	.015	.016						.316
E-2						.015	.014	+.001	.006	.045	.057	.023	.021	.180
F-8						.020	.019	.000	.009	.025	.050	.022	.021	.166
D-2						.033	.025	.007	.014	.038	.069	.039	.028	.253
F-7						.039	.029	.006	.020	.041	.079	.043	.032	.289
C-2						.039	.030	.010	.019	.053	.091	.043	.033	.318
F-6						.043	.034	.013	.021	.060	.105	.047	.033	.356
B-2						.040	.031	.012	.023	.068	.110	.061	.042	.387
F-5						.045	.034	.013	.027	.078	.123	.067	.048	.435
A-4						.041	.033	.015	.027	.090	.138	.068	.051	.463
A-3						.037	.032	.014	.032	.107	.156	.084	.058	.520
F-4						.042	.034	.015	.033	.113	.166	.084	.059	.546
A-2						.039	.030	.014	.029	.117	.169	.079	.058	.535
A-1						.038	.025	.008	.026	.112	.168	.081	.056	.509
B-1						.033	.020	.007	.019	.101	.146	.064	.045	.435
C-1						.028	.016	.007	.015	.075	.113	.055	.035	.344
F-3						.029	.017	.003	.014	.066	.102	.048	.030	.309
D-1						.026	.013	.006	.008	.040	.072	.027	.019	.211
F-2						.025	.013	.003	.008	.028	.062	.021	.013	.173
E-1						.014	.004	+.001	.006	.005	.030	.010	.005	.073
F-1						.013	.001	+.003	.007	.000	.021	.005	.001	.045

toward the center as the dam arch flattens. The comparative readings between some monuments seems to be questionable. Some check readings taken during the compilation of this data tends to bear out the magnitude of these readings based on the rate of change over a short period. An additional measurement system for this type of records has now been set up, with the hope of determining whether this inconsistency does exist or if the initial positions are in error axially.

Shortly before midnight on August 17, 1959, a severe earthquake occurred in the Hebgen dam area in Montana. The epi-center of this quake was reported to have been at a point approximately 300 miles from Brownlee dam. Intensity at the epi-center was variously estimated at 7.2 to 7.8 on the Richter scale. On August 18 readings were taken on all monuments and settlement rods along the evest of Brownlee Dam. Those readings did not reveal any effect of the earthquake on the rate of movement.

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ROCKFILL DAMS: SALT SPRINGS AND LOWER BEAR RIVER CONCRETE FACE DAMS[a]

Discussion by C. M. Roberts

C. M. Roberts.[1]—An engineer faced with the problem of providing a rockfill dam now has available to him a wealth of information on which to base his design. Notably there are the excellently presented descriptions of the design, construction and performance of the Salt Springs Dam offered by the authors. As a result of the extensive data provided by them, the Salt Springs Dam has set a pattern for numerous subsequent concrete faced rockfill dams throughout the world.

The lessons of experience will however have best been learned by those closely in touch with the original work and their latest paper, in showing how the authors have themselves developed further designs and construction techniques to improve on the Salt Springs Dam, is therefore invaluable.

Rockfill dams have perhaps been adopted less frequently in Europe than elsewhere for the reason that concrete dams can be constructed more economically and not for any reasons of prejudice. In fact, a rockfill dam only becomes justifiable when a large proportion of the fill may be taken from other essential excavations such as a tunnel in a hydro project. This was one of the factors that made consideration of a rockfill dam possible for the Quoich reservoir in Inverness-shire, Scotland.

This dam has been fully described in a paper by the writer to the Sixth Congress on Large Dams in New York, 1958,[2] but a few remarks appear pertinent in this discussion to contrast the placing and consolidation methods used there with the dump fill method. A typical cross-section of the Quoich dam is shown in Fig. 1.

The Quoich reservoir is intended for long-term storage and it was essential that the dam should not be subject to any inherent weakness which might necessitate drawing down the reservoir for maintenance or repair work. Accordingly special measures were introduced and in particular, careful attention was given to the consolidation of the dam hearting.

As a result of discussions with the Swedish Engineers V.B.B., the writer was convinced that a rockfill dam could be constructed having virtually negligible settlement after completion of the fill. The method involved required the placing of the fill in 2 ft. thick layers and the anticipated use of tunnel spoil with consequent limited rock particle size made this a feasible proposition, permitting also the use of conventional compaction techniques. The

a. Proc. Paper 1737, August 1958, by I. C. Steele and J. B. Cooke.

1. Partner, Sir William Halcrow & Partners, London, England.

2. The Garry & Moriston hydro-electric schemes." Proc. I.C.E. Sept. 1958.

9'0" roadway
Flood level 668 000 at centre of dam
Flood level 664 000 a
Top water level 660 000 a
Slope 1 in 1 4
9" dry pitching
Max height 126 ft approx
24" dia open jointed concrete pipe
General line of steps sloping 1 in 1 2 at ends of dam and 1 in 1 82 at centre
Face of hearting formed in steps from 564 000
Max width 340 ft approx
Fall 1 in 24
Hearting of tipped rockfill consolidated and sluiced in layers approx 2'-0" thick, after dust and fines below ¾" dia had been removed
Slope of upstream face 1 in 1 30 at ends of dam varying to 1 in 1 28 at centre
Hand packed rubble
Upstream facing slabs 20'-0" x 20'-0" generally. Slabs 12" thick above 568 000 and 15" thick below.
Sound rock
Holes for fissure grouting at 5ft staggered centres.
Fine backfill
Cut off trench
3'-0"

QUOICH DAM TYPICAL CROSS SECTION

speed of construction was not a significant factor as the time consideration was dictated by the driving of the power tunnel.

Concern was raised by the presence of micaceous bands in the schists traversed by the tunnel giving rise to excessive fines in the tunnel spoil. A system of inspection of the tunnel face was instituted whereby a visual assessment of the proportion of mica in the rock was made prior to the firing of each round. If this was excessive the rock from that pull was considered unsound and was rejected for use in the dam. Generally however the spoil was merely passed through a simple dirt extraction plant which removed fines below 3/8", some 16.4% of the spoil being eliminated in the process.

The clean rock was tipped onto the dam by trucks and spread by bulldozer to the required depth of 2 ft. and cross slope of 1 in 24. The passage of these vehicles over the fill gave a substantial degree of compaction before any formal consolidation measures were carried out. The first consolidation operation was rolling with a 10 ton smooth roller. In addition to pure compaction this process levelled out the surface, breaking off any upstanding points of rock, thus preparing for the second operation which was vibration with a 3-1/2 ton vibrating roller. Tests had established that vibration gave better consolidation than could be obtained with plain rolling. The vibrating roller was most effective when operating near the upper part of its frequency range i.e. 1500 cycles/min. As it approached this frequency a state of resonance occurred and the surrounding area of rockfill would shake with the vibration.

Sluicing of the surface of the fill was carried out after each stage of the compaction process to wash down into the body of the fill any fines caused by the rolling. Trial pits dug into the compacted rockfill showed that significant fines were formed but that the fill generally was compacted into a tight mass having only some 27.1% voids. The successive layer surfaces could not be distinquished in the trial pits.

The placing of the fill in layers permitted close control of the upstream and downstream slopes of the fill. In fact the upstream slope at the junction between the hearting and the hand placed stone was steeper than the natural slope and had to be supported by steps of hand placed stone set as the fill proceeded.

The effectiveness of the compaction procedure may be judged from the measured settlement after completion of the dam. The measurement points are described in the paper previously referred to. The latest observations show that the maximum settlement or deflection nowhere exceeds 3/4 inch.

In view of this result it appears that certain relaxations might be considered if this type of dam were to be repeated. The removal of fines prior to placing of the fill would be a question for judgment depending upon the nature of the rock involved. The sluicing of the fill has less significance than with the dump fill method and might be reduced. The thickness of hand placed rubble could be reduced and the sizes of the individual panels forming the concrete membrane might be increased.

ROCKFILL DAMS: SALT SPRINGS AND LOWER BEAR RIVER CONCRETE FACE DAMS[a]

Closure by I. C. Steele and J. B. Cooke

I. C. STEELE,[1] F. ASCE and J. B. COOKE,[2] F. ASCE.—The authors thank Messrs. Carlos Tercero, Geoffrey Davey, Masatoshi Kawase, Tatsuo Mizukoshi, F. L. Lawton, and C. M. Roberts for their discussions which emphasize important points and present valuable and concise data on six other concrete face rockfill dams.

The 189-ft. high Pinzanes Dam, described by Mr. Tercero, is certainly very successful as well as economical with its thin placed rock and steep slopes. It is satisfying that the published experience of Salt Springs Dam was a factor in the design and that the use of sloping joints, as suggested by experience at Salt Springs Dam and as used at Lower Bear River Dam No. 1, was also successful at Pinzanes Dam. The face crack above the wet masonry at Pinzanes Dam repeats the adverse experience at Salt Springs, as mentioned in the paper, and confirms that wet masonry should not be used in lieu of placed rock above the cutoff line. That the dam was not affected by the July 28, 1957 earthquake is noteworthy. The method referred to by Mr. Tercero, used by P. G. and E. Company to locate the point or points of leakage without unwatering the reservoir, is an audio method which is based on the sound of water entering an opening in the face of the dam. Having located a leak, it can be plugged underwater by depositing a mixture of bentonite, sand and drillers cellophane; or by the work of a diver.

Mr. Geoffrey Davey raises two important questions: (1) the influence of height of lift on settlement; and (2) to what extent is the possibility and economic consequences of leakage a factor in selection of a concrete face type of rockfill dam.

Compaction of a rockfill dam can be in the form of layers of about 1.5 to 5 feet or in the form of lifts of about 25 ft to 200 ft. The thin layers are used where the rock sizes are predominately small, and in the highest core rockfill dams in zones adjacent to the transition. The placing of rockfill in compacted layers adjacent to the transition zones of a high core rockfill dam would be to reduce the size of voids as well as to minimize settlement. The selection of the adequate and economic thickness (about 1.5 to 5 ft.) of the layers depends to a large extent on the grading of rock, and on the importance of minimum settlement. Unless the rock is very small, cost would increase as the layers are made thinner. An example of the small settlement of rockfill compacted in layers is provided by the 124 ft high Quoich concrete face

a. Proc. Paper 1737, August 1958, by I. C. Steele and J. B. Cooke.
1. Cons. Engr., San Francisco, Calif.
2. Superv. Civ. Engr., Pacific Gas and Electric Co., San Francisco, Calif.

rockfill dam which showed no measurable settlement upon the reservoir filling,(1) and a total maximum crest settlement of only 3/4 inch after four years of service.

Construction of rockfill in lifts rather than compacted layers is particularly economical and is considered to be appropriate where the grading of rock includes a number of rocks in the range of 2 to 20 or more tons in weight. There are differences of opinion on the effect of height of lifts on settlement of a completed dam. The factual data on construction and settlement presented in this symposium and in the future will tend to provide a better answer to the question of the effect of height of lift on settlement of a completed dam, and also to the question of the relative settlement of a rockfill constructed in layers as against one constructed in lifts.

The concrete face rockfill dam is, of course, subject to probable leakage. The significance of this factor is believed to be minor in the economics of selecting the type of dam. Where a concrete face dam is the type of dam most economical and appropriate to the site, the capital cost saving is usually so great that leakage problems or maintenance costs are nominal factors. The first reservoir filling, or possibly the second, should see the end of any movements or stresses that would cause leakage to occur. The leakage may be located and stopped by detection and repair either underwater or in the dry. After initial repair, should it be necessary, there should be no significant future trouble.

Mr. Masatoshi Kawase and Mr. Tatsuo Mizukoshi have presented valuable data on the 174-ft. high Ishibuchi and the 144-ft. high Nozori concrete face rockfill dams. The two dams are similar to the 150-ft. high Lower Bear River Dam No. 2. The construction record shows that the rock in these three dams was dumped in high lifts and about equally well sluiced. The settlements have been similar in all respects, with maximum vertical crest settlement of about 0.25% of height of dam and on the order of 0.3 feet. The performance of all three has been particularly good from the standpoint of moderate settlement and moderate face movement; and from the standpoint of little or no leakage and maintenance.

Mr. F. L. Lawton has emphasized important points in the paper, among which is the similarity that should exist between settlement in concrete face and sloping core rockfill dams. The $2,000,000 capital cost saving of the rockfill dam over a concrete dam at Salt Springs compares with only $88,000 ($8,000 per year capitalized at 11%) that could have been spent on the concrete face dam to eliminate maintenance, or that could have been spent toward a type of dam having no maintenance. The 11% fixed charges includes about 2.5% local taxes and 2.5% federal taxes.

Mr. C. M Roberts has presented a concise discussion on the economical and most successful Quoich Dam. His inferred conclusion that the dam was perhaps too good and his stated conclusion that economies perhaps could be made in repeating such a dam are significant and important. The writers came to a similar conclusion based on the excellent performance of the Lower Bear River Dams and major economies were effected in the subsequent Wishon and Courtright Dams.(2)

Whereas Quoich Dam was made economical by the use of tunnel muck, the economics of many rockfill dams is greatly improved by laying out the spillway as a main quarry. Though such economies in utilizing essential excavations often result in the economic selection of rockfill type dams, such is not always the case as evidenced by the selection of the sloping core rockfill type

for the Kenney site where all materials were specially quarried for the dam. Tunnel muck is necessarily compacted in layers but rock from a spillway quarry would generally contain enough large rocks to permit placing in lifts.

The Swedish experience, referred to by Mr. Roberts, was particularly in relation to the 50-foot high concrete face Nissastrom Dam(3) which was constructed of tunnel muck placed in layers, sluiced, rolled and vibrated. The settlement after completion of the rockfill and after filling was negligible for both Nissastrom and Quoich Dams. The two dams are not high and the compaction, as noted by Mr. Roberts, was probably more than necessary. However, the primary value of the compaction experience, as thoroughly presented in references (1) and (3), is in its possible application to zones adjacent to the cores of core rockfill dams of major or unprecedented height. The effective use of vibration equipment on rockfills is of particular interest. The 27.1% void figure for Quoich Dam compares with 29 to 30.7% voids for the similarly compacted tunnel muck at Nissastrom Dam, and with 30 to 32% voids for a number of dumped rockfills of larger rock size.

The bar graph of Fig. 7 of the paper shows the vertical settlement of crest point H to be 0.09 feet per year between the 18th and 27th year and the total to be 3.6 feet. This rate of 0.09 feet per year seemed inconsistent with the 0.05 feet per year for the previous four years. A careful check of the notes and of the movement of adjacent points, together with a recent reading, revealed an error of 0.30 ft. Figure 7 should read 0.055 feet per year for the 18th to 27th year and 3.3 feet for the total at the end of 1957. Correspondingly, the 3.6 feet shown on Fig. 6 should be 3.3 feet. The end of 1959 reading gives 3.34 feet total vertical settlement for 29 years and a rate of 0.02 feet per year for the last 2 years.

A similar correction should be made in the text on Page 17 of the paper in connection with Salt Springs settlement. The sentence on the crest settlement, Point H of Fig. 7, should read, "The crest, Point H of Fig. 7, has settled vertically after the first two years at a rate that has decreased from 0.10 feet per year to 0.02 feet per year in 29 years." This is an important correction.

In comparison with the 275-ft. high Dix River Dam, constructed of limestone rock, the higher 328-foot high Salt Springs Dam, constructed with granite rock, has settled less and reached a more stabilized condition. Dix River Dam vertical crest settlement has been 4.22 feet in 32 years and has been at a rate of 0.07 feet per year between the 25th and 30th year and also at a rate of 0.07 feet per year for the 30th to 32nd year. Possibly the difference in the stabilized rates of settlement for Salt Springs and Dix River Dams is due to the different characteristics of the rock in the two dams.

A typographical error on Page 17 of the paper showed a decimal point in the wrong place for several numbers and should be corrected to read, "At Point F, located 92 feet lower than the crest, the vertical rate of movement has decreased from 0.076 to 0.044 feet per year, but has been constant for the last 13 years at 0.044 feet per year, a little less than the 0.055 feet per year for Point H."

At the end of 7 years of service, the leakage from Lower Bear River Dam No. 1 is still less than 2.0 cfs, and the leakage from Dam No. 2 is still zero. No maintenance has been required during the 7 years. At the time of writing the paper, data on settlement for only the first 4 years of service was available. The crest settlement for the 7-year period to the end of 1959, is presented in revisions of Table III and of the table in Fig. 21 of the paper. No

TABLE III.

REVISED TO EXTEND RECORD FROM 4 TO 7 YEARS

LOWER BEAR RIVER DAM NO. 1												
TABLE OF CREST SETTLEMENT - IN FEET												
STA.	HT. AT AXIS	*	YEARS								% OF HT.	
			1 ST	2ND	3RD	4TH	5TH	6TH	7TH	7YRS	1YR	7YRS
1+40	110	V	.77	.02	.02	.05	.017	.017	.017	.91	.70	.83
		H	.57	.00	.07	.00	.023	.023	.023	.71	.52	.65
3+20	220	V	1.10	.03	.05	.05	.030	.030	.030	1.32	.50	.60
		H	.74	.06	.07	-.01	.020	.020	.020	.92	.34	.42
4+40	220	V	1.03	.03	.04	.06	.030	.030	.030	1.25	.47	.57
		H	.86	.02	.07	.01	.030	.030	.030	1.05	.39	.48
5+60	150	V	.89	.02	.05	.02	.020	.020	.020	1.04	.59	.69
		H	.76	.03	.02	.01	.030	.030	.030	.91	.51	.61
6+80	85	V	.23	.01	.02	.03	.007	.007	.007	.31	.27	.36
		H	.22	.01	.03	.00	.010	.010	.010	.29	.26	.34

* V - VERTICAL COMPONENT H - HORIZONTAL

TABLE OF FIG. 21

REVISED TO EXTEND RECORD FROM 4 TO 7 YEARS

LOWER BEAR RIVER DAM NO. 2												
TABLE OF CREST SETTLEMENT - IN FEET												
STA.	HT. AT AXIS	*	YEARS								% OF HT.	
			1 ST	2ND	3RD	4TH	5TH	6TH	7TH	7YRS	1YR	7YRS
13+95	85	V	.23	.03	.00	.00	.007	.007	.007	.28	.27	.33
		H	Δ.23	.03	.02	.03	.007	.007	.007	.33		
15+75	140	V	.29	.04	.01	.01	.013	.013	.013	.39	.21	.28
		H	Δ.29	.04	.04	.01	.007	.007	.007	.40		
17+55	120	V	.19	.04	.00	.01	.007	.007	.007	.26	.16	.22
		H	Δ.19	.04	.03	.01	.01	.01	.01	.30		
18+75	80	V	.11	.02	.00	.01	.003	.003	.003	.15	.14	.19
		H	Δ.11	.02	.00	.04	.007	.007	.007	.19		

* V = VERTICAL COMPONENT H = HORIZONTAL

Δ THE 1ST YEAR HORIZ. READING WAS ESTIMATED AND ASSUMED TO BE EQUAL TO THE VERT. READING SINCE NO INITIAL HORIZ. READING WAS MADE PRIOR TO FILLING RESERVOIR.

readings were taken between the 4th and 7th year and the movements of the 5th, 6th, and 7th years are based on assumed equal movement each year. For the 5th through the 7th years, the maximum vertical crest settlement has been at a rate of 0.03 feet per year for Dam No. 1 and 0.01 feet per year for Dam No. 2. This data gives concrete evidence of the early stabilization of settlement to a negligible amount for a rockfill dam of competent rock that is well sluiced and dumped from high lifts.

The notable paper by the late Mr. J. D. Galloway, with its discussion by a number of engineers,(4) is an important document in the history of rockfill dams. That paper in 1939 developed and stimulated interest in rockfill dams and constituted an early major contribution to the technology of rockfill dams. The papers of the ASCE Symposium on Rockfill Dams, along with the discussions by numerous engineers throughout the world, comprise a major step toward still further progress in the field of design and construction of rockfill-type dams.

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1. The Quoich Rockfill Dam, by C. M. Roberts, Paper R. 6, Sixth International Congress on Large Dams, October 1958.

2. Rockfill Dams: Wishon and Courtright Concrete Face Dams, by J. Barry Cooke, Proceedings of the American Society of Civil Engineers, Paper 1746, August, 1958.

3. Compaction of a Rockfill Dam, by Dr. B. Hellstrom, Paper R. 35, Fifth International Congress on Large Dams, Paris, 1955. Paper R. 35 is also available as Bulletin No. 47, Royal Institute of Technology, Stockholm, Sweden.

4. "The Design of Rockfill Dams," by J. D. Galloway, Transactions of the American Society of Civil Engineers, Vol. 104, 1939. With Discussions by: Cecil E. Pearce, H. B. Muckleston, Harold K. Fox, Charles H. Paul, A. Floris, Howard F. Peckworth, Oren Reed, Walter L. Huber, Samuel B. Morris, L. F. Harza, Paul Baumann, O. W. Peterson, George W. Howson, John E. Field, John H. Wilson, Frederick H. Fowler, I. C. Steele and Walter Dreyer, F. Knapp, Ralph J. Reed, F. J. Sanger, C. S. Jarvis, L. R. East, Francisco Gomez-Perez, Miguel Jinich, and J. D. Galloway.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867(ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876(WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926(PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998(SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007(HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

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JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087(SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096(EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

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OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330(CP2).

c. Discussion of several papers, grouped by divisions.

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NEWS

December, 1959

The Committees for 1960 are:

Executive Committee

Chairman	Marcel P. Aillery	(1962)
Vice-Chairman	Robert T. Colburn	(1962)
Other Members	George J. Vencill	(1960)
	John F. Bonner	(1961)
Secretary	Richard R. Randolph Jr.	
Contact Member on Board of Direction	Charles W. Britzius	

Division Administrative Committees

Committee on Publications - Chairman	Thomas J. Cotter
" " Research "	Marcel P. Aillery
" " Rickey Medal "	Joel P. Justin
	Mr. D. J. Bleifuss has joined the committee in lieu of Adolph Ackerman whose term expired.
" " Session Programs (Revolving chairmanship)	Robert J. Drueding J. G. Thon R. W. Gunwaldsen

Division Technical Committees

Committee on Control of Cracking in Masonry Dams
Chairman Kenneth B. Keener

Committee on Nuclear Energy " Richard N. Bergstrom
Messrs Gordon L. Coltrin, Robert M. Jacobs and Gordon H. Osterheld were added to the committee and Mr. F. F. Mautz resigned.

Committee on Operation & Maintenance of Hydro-electric Generating Stations Chairman Karl O. Strenge

Committee on Progress in Power Plant Design - Glenn H. Von Gunten

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Representation on Committees of the Society

Water Resources Co-ordinating Committee:
Representative: George J. Vencill

Committee on Technical Procedure:
Representative: Marcel P. Aillery

Cooperation with other Societies
American Power Conference Representative: Richard N. Bergstrom

ASME Hydraulic Prime Movers Subcommittee
Representative: Robert A. Sutherland

Excerpts from Annual Report 1958-59 - Members in division 1509
Members of Committees & sub-committees 26
No. of committee reports 4
No. of Div. Technical sessions 12
No. of papers presented at Division sessions 38
Papers published - 20 papers in six Journals

1959 Annual Meeting - The Power Division was well represented at the Annual Meeting in Washington, D. C. Oct. 19-23, 1959 with 16 papers on the work of engineers abroad.

Future Meetings - Winter Convention March 7-11, 1960 at New Orleans, La. Mr. Robert J. Drueding, 4455 Lafaye St. New Orleans, La. is program chairman

The tentative program, of which the first session will be jointly sponsored by the Structural Division is:

Session 1 - Preliminary Design of Guyed Towers - by Robert S. Rowe
Design of Self-supported steel Transmission Towers - by Richard N. Bergstrom
Transmission Line construction by Helicopter - by Frederick R. Payne

Session 2 - Little Gypsy Power Plant - by Geo. A. Bingham
Power Plant on a Platform at sea - by H. R. Davies & T. H. Tumwalt
The Design of Arch Dams by the Trial Load Method of Analysis by M. D. Coper
Use of Digital Computer in Stress Analysis of Arch Dams - by L. R. Scrivner

Session 3 - Lock 17 Power House, Warrior River Development - by Gale B. Dougherty & R. R. Randolph
Coosa River Hydro Projects - by Richard S. Woodruff
Multi-Purpose Power Plant Capacity - by Richard E. Krueger

A field trip to the 220 Mw Little Gypsy power plant will be arranged, a feature of special interest being that this plant will be completely automatic.

Summer Convention - June 21-25, 1960 at Reno, Nevada

Mr. J. G. Thon, 465 Barbara Way, Hillsborough, Calif. is program chairman, and joint sessions with the U.S. Committee on Large Dams are planned.

Annual Convention - Oct 11-15, 1960 at Boston, Mass.
Mr. Ralph W. Gunwaldsen, 42 Temple Road, Wellesley, Mass. is program chairman.

Spring Convention 1961 - April 10-15, 1961 at Phoenix, Arizona

Members interested in taking part in any of these programs should contact the local program chairman or any member of the Executive Committee.

Expanded Scope of Committees. Consequent on a recommendation by the Society's Water Resources Coordinating Committee and its acceptance by the Executive Committee the duties of three of the technical committees have been expanded as follows:

Committee on Nuclear Energy is assigned the additional problems of water uses for nuclear plants and the disposition of radioactive wastes.

Committee on Progress in Power Plant Design is assigned the additional study of sources of water for power utilization and the technique of pumped storage.

Committee on Operation and Maintenance of Hydro-electric Generating Stations has been assigned the additional subjects of a) flooding on upstream lands b) regulation of flow through power pools and c) relation of power impoundments to recreation and wild life.

ASME Papers of Interest The following papers to be presented at the ASME annual convention on Nov. 30 and Dec. 2, 1959, may be of interest to members and may be obtained from ASME by ordering the Paper No. (where given).

Charts for Determining Size of Surge Suppressors for Pump Discharge Lines; by C. W. Lundgren (59A-73)
Surge Tank Stability and the Validity of the Formula of Thoma; by C. Jaeger
Vibration of Water Turbine Draft Tubes; by M. Murakami (59-A-96)
Reversible Pump Turbines at Niagara Falls; by F. E. Jaski & W. W. Weltmer (59-A-87)
A New Runaway Speed Limiter for Kaplan Turbines; by G. H. Voaden (59-A-103)
Simplified Instrumentation and Field Checking of Hydraulic-Turbine Governors; by B. E. Wheeler (59-A-124)
Field Testing and Adjusting of Hydraulic Turbine Governors to Improve System Regulation by H. M. Stone (59-A-149)

Field Adjustment of Hydraulic Turbine Governors;
by C. L. Avery (59-A-108)
Some Corrosion Effects in Accelerated Cavitation Damage;
by W. C. Leith (59-A-52)
On Cathodic Protection in Cavitation Damage;
by M. S. Plesset (59-A-170)
Design of Suction Piping & Deaerator Storage Capacity to Protect Feed Pumps; by R. S. Thurston (59-A-20)
Centrifugal Pumps used as Hydraulic Turbines;
by C. P. Kittredge (59-A-136)

Passing of Col. F. W. Scheidenhelm The Power Division has lost a stalwart supporter and former officer of the Division by the death on October 17, 1959 of Colonel F. W. Scheidenhelm consulting engineer of New York. During the last 2 years, he alone or in cooperation with others had contributed two valuable papers on rockfill dams to the Division, and he had during his whole professional career given freely of his time and energy to the profession.

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